

Characterization and stabilization of Eastern Saudi Marls

Habib-ur-Rehman Ahmed

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Abstract

Calcareous soils, locally known as marls, are extensively exploited in the construction of highways and building foundations. Despite their wide prevalence all-over the world, a review of the literature indicates that calcareous soils have received little attention from the geotechnical community. Therefore, this comprehensive research program was devoted to study first the characterization and the geotechnical properties of eastern Saudi calcareous soils.

Marl sample were collected from over 20 different sites in the Eastern Province of Saudi Arabia. The samples were characterized and their geotechnical properties were determined. Based on these tests, it was found that marl soils are extremely sensitive to the molding and testing moisture contents. The existing classification systems and specifications are of limited applicability to these soils classification systems and specifications are of limited applicability to these soils.

Owing to their poor engineering behavior, the second phase of this research program focused on improvement and stabilization of some selected marl soils. The selection was based not only on the poor behavior of these soils, but also the availability of these soils and their potential future use. Mechanical and chemical soils stabilization techniques were investigated. Chemical stabilization using cement was found to be an effective and efficient way to significantly improve the inferior properties of these soils in terms of strength and durability. This research has opened a new door towards a better and more economical utilization of the low quality marl soils in eastern Saudi Arabia in particular, and calcareous soils worldwide in general.

Characterization and Stabilization of Eastern Saudi Marls

by

Habib-ur-Rehman Ahmed

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS

DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

In

CIVIL ENGINEERING

December, 1995

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COLLEGE OF GRADUATE STUDIES


This Thesis, written by


Habib-ur-Rehman Ahmed

under the direction of his Thesis Advisor, and approved by
his Thesis Committee, has been presented to and accepted
by the Dean of the College of Graduate Studies, in partial
fulfillment of the requirements for the degree of


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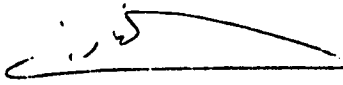
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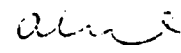

Dr. Saad A. Aiban
(Chairman)


Dr. Omar S. B. Al-Amoudi
(Co-Chairman)


Dr. Hamad I. Al-Abdul Wahhab
(Member)


Dr. Naser A. Al-Shayea
(Member)


Dr. Al-Farabi M. Sharif
Department Chairman


Dr. Ala H. Al-Rabeh
Dean, College of Graduate Studies

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Dated



This Humble Work is Dedicated to:

**To my esteemed parents,
who elegantly extended their
affectionately-oriented, unparallel
devotion, all the time, for me.**

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Abstract

Name: Habib-ur-Rehman Ahmed
Title: Characterization and Stabilization of
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Calcareous soils, locally known as marls, are extensively exploited in the construction of highways and building foundations. Despite their wide prevalence all-over the world, a review of the literature indicates that calcareous soils have received little attention from the geotechnical community. Therefore, this comprehensive research program was devoted to study first the characterization and the geotechnical properties of eastern Saudi calcareous soils.

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King Fahd University of Petroleum and Minerals
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الخلاصة

الإسم : حبيب الرحمن أحمد

العنوان : تقييم وتثبيت التربة الجيرية الموجودة في شرق المملكة العربية السعودية

القسم : الهندسة المدنية

التاريخ : ديسمبر ١٩٩٥ م

تستخدم التربة الجيرية المعروفة محلياً بـ " المارل " في إنشاء الطرق وفي أساسات المباني . وعلى الرغم من تواجد هذه التربة في جميع أنحاء العالم فإن دراستها والإهتمام بها من الناحية الجيوتقنية كانت قليلة . ولذلك فإن هذا البحث المستفيض قد خُصص للدراسة الخواص الجيوتقنية للتربة الجيرية المتواجدة في المنطقة الشرقية من المملكة العربية السعودية .

وقد تم جمع عينات التربة الجيرية من أكثر من عشرين موقعاً مختلفاً في المنطقة الشرقية . وتم تحديد الخواص الجيوتقنية المميزة لكل تربة . وقد دلت النتائج على أن خواص التربة الجيرية ذات حساسية عالية للماء أثناء الخلط وكذلك عند الاختبار . كما دلت النتائج على أن أنظمة التصنيفات المعروفة للتربة لا تنطبق على التربة الجيرية إلا بشكل محدود .

ونظراً للخواص الهندسية المتدنية لهذه التربة ، فإن المرحلة الثانية من هذه الدراسة تتركز حول تطوير وتحسين بعض أنواع هذه التربة والتي تم إختيارها بناءً على المستوى المتدني من ناحية وتواجد هذه الأنواع بكثرة من ناحية أخرى حيث يمكن إستخدامها في المشاريع الإنشائية المستقبلية .

وقد تم دراسة الطرق الميكانيكية والكيميائية لتحسين هذه التربة ، حيث دلت النتائج على فعالية إستخدام الأسمنت في تحسين القوة والتحمل لهذه التربة . وهذا البحث قد فتح المجال لإستخدام المواد المتوفرة محلياً - ذات المواصفات الغير جيدة - بطريقة إقتصادية ملائمة ، وذلك بعد تحسينها . وهذا ينطبق على التربة الجيرية في شرق المملكة العربية السعودية على وجه الخصوص والتربة الجيرية على مستوى العالم بشكل عام .

درجة ماجستير

قسم هندسة المدنية

جامعة الملك فهد للبترول والمعادن

الظهران ، المملكة العربية السعودية

ديسمبر ، ١٩٩٥ م

Chapter 1

Introduction

Marl is defined as a soil or rock-like material containing 35 to 65% calcareous material and a complementary content of clay (Pettijohn, 1975). The term marl is used to represent all types of calcareous soils present in the Eastern Province of Saudi Arabia. Locally, these soils are being used in the construction of all types of pavements and foundations. Owing to their poor performance, which depends on several factors, the engineering properties of these soils need to be explored in detail. The exploration should consider the inferior properties of these soils and the possible means of improving these properties.

1.1 Calcareous Soils

The word calcareous means “containing, or partly composed of, calcium carbonate (CaCO_3)” (Challinor, 1978). Calcareous soils exist in many parts of the world, and they are not only found on land but also on the sea-floor. According to Horta (1988), Yaalon estimated that calcareous soils cover about 13% of the total area of the emerged land. Approximately, 36% of the deep sea-floor is blanketed by a thick layer of clay to fine sand-size remains of calcareous marine life, known as the “calcareous ooze” (Valent et al., 1982). These soils possess distinct geotechnical

properties which are quite different from those known in classical soil mechanics (Beckwith and Hansen, 1982).

The calcareous soils show abnormal properties both in their natural conditions and when used in construction. These properties result from the constituents of the soils themselves and the mode of formation. The main constituent is calcium carbonate, which can be of organic or inorganic origin. In either case, the calcium carbonate particles cannot be treated as normal soil particles. The distinctive behavior of these soils makes the applicability of certain standard testing procedures doubtful. The factors affecting the engineering properties at the time of formation are the deposition environment in the case of sedimentary calcareous soils, and the type of host soil in the case of non-sedimentary calcareous soils. These factors lead to variation in the calcareous soil deposits in terms of their constituents. The variability exists in both the lateral and the vertical directions.

Calcareous soils found in the Eastern Province of Saudi Arabia, similar to those in other parts of the world, contain many impurities (Aiban, 1994). These soils exhibit wide variations in their characteristics, engineering properties and even in their definition. This is mainly due to: (1) the presence of different types of impurities, such as gypsum, aragonite, calcite, dolomite, sand, chert, quartz, geodes, etc., (2) layering of the borrow pits, and (3) their burial with detritus sediments. Moreover, the properties of these soils are observed to be time-dependent due to their extensive layering (Aiban, 1995).

1.2 Engineering Behavior of Calcareous Soils

The behavior of calcareous soils is complicated and their engineering properties are affected by many factors such as: the carbonate content, degree of induration, particle size; origin of carbonate material, content of non-carbonate material, environmental conditions and the construction procedures. Typical deviations in the behavior of these soils, as compared to the non-carbonate soils, include particle crushing, cementation, negative shrinkage indices, non-plastic carbonate fines, low friction sand-sized particles, large differences between the unwashed and washed grain-size distribution, solubility of the carbonate fraction in water, self-stabilization and recrystallization, poor gradation, intraparticle water leading to erroneous results for the Atterberg limits, and the most crucial factor is their acute water sensitivity (Horta, 1980; Horta, 1988; Netterberg, 1967; Netterberg, 1982). The water sensitivity can lead to a complete collapse and reduction in the bearing capacity when these soils get into contact with water. This is mainly attributed to the loss of cohesion of frictionless material, and the reduction of friction of the cohesionless material.

The sand-sized particles do not generally contain silica in most of the cases. The calcite, dolomite, and other non-silica particles of marine origin, offer low friction and are susceptible to crushing upon loading and dissolution. When excess water comes into contact with these soils, a very weak matrix of water and fine particles is formed in which the gravel fraction just float. Therefore, the granular-bearing skeleton which has a very high friction on the dry side of optimum of the compaction curve, forms a mud-like structure in the presence of water. Fur-

ther, the presence of aggregate-sized particles of gypsum, calcite, etc., are prone to crushing, dissolution and loss of stability when wet. Consequently, these soils are not suitable to be used in construction projects associated with high water tables or exposed to heavy rainfalls unless these soils are initially stabilized. In relatively dry areas, these soils behave adequately. Therefore, the existing specifications are of limited use because no consideration is being given to the water susceptibility of these soils when used in construction. As a consequence, engineers are using their own specifications and judgment when dealing with such soils. According to Horta (1988), the minimum value of plasticity index (PI) is increased from 6% to 15% for the use of these soils as a base material. Calcium carbonate equivalent (calcium carbonate content of the soil passing ASTM #40 sieve) is being widely used as an index instead of the PI, since it is more representative of the behavior of these soils. Field Pliers test is also used to give an idea about the hardness of the gravel fraction of these soils (Netterberg, 1967). The hardness is measured by crushing the aggregates using a standard pair of pliers. The hardness or resistance to abrasion of the gravel fraction is one of the controlling factors for the use of these soils in highways. Hard and sound aggregates, in most cases, minimize the negative effects of calcareous soils.

Due to their low strength and water sensitivity, there have been attempts to stabilize and improve the engineering properties of calcareous soils using different additives such as cement, lime, emulsified asphalt and cut-back asphalt. Most of the chemicals used gave results with various degrees of success; and the stabilized soils can be recommended for use as base and/or sub-base and for building foundations or other applications even when water can reach these soils. In addition,

the carbonates present in these soils are soluble in acidic water and consequently precipitate when water evaporates. This may have an advantage of providing cementation to the compacted soils, which can be utilized for the self-stabilization of the soil. However, the water sensitivity of these soils is still not being investigated thoroughly. Therefore, there is an urgent need for an engineering exploration and stabilization of these soils from construction prospectives.

1.3 Objectives

Calcareous soils are one of the predominant soils found in the Eastern Province of Saudi Arabia. They are extensively utilized as foundation materials for structures and pavements. The presence of gypsum, anhydrite, aragonite, calcite, expansive clay, sand, chert, quartz, geodes and other impurities in most of the marl sediments makes these soils behave in an erratic way. Various types of calcareous soils virtually exhibit greatly different characteristics from one location to another in terms of physical and chemical properties. Even within the same site, vertical and horizontal variations are frequently observed. The lack of published and thorough information on the engineering properties and performance of these soils under the harsh environmental and loading conditions has led to speculation and wide discrepancies in the behavior and usage of these soils.

Another important aspect of these soils is their acute sensitivity to both molding moisture content and in-service exposure to flooding. Extensive alligator cracks are frequently observed, even in the lightly trafficked and newly constructed roads, especially when these soils are used as a base material in areas where the water table is high. Exemplary case studies are depicted in Fig. 1.1. The presence of

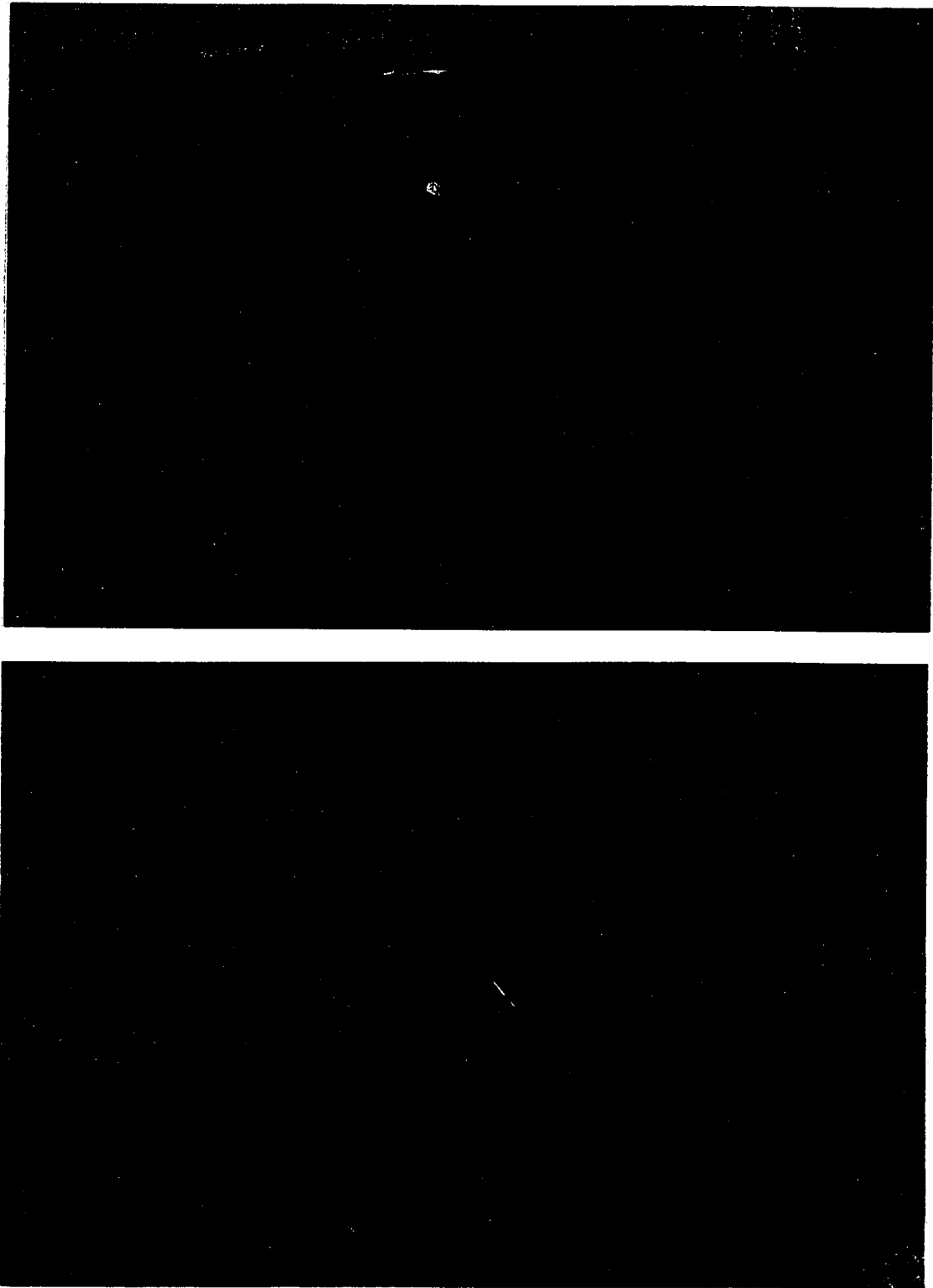


Figure 1.1: Typical Deterioration of a Newly Constructed Road on a Marl Base in Dammam, Eastern Saudi Arabia.

such a problem indicates a substantial reduction in the bearing capacity of the supporting layers. Hence, there is an urgent need to stabilize these soils prior to any construction. Although there are some pilot studies on blending these soils with sand and asphalt, the results obtained cannot be generalized due to the variability of marl soils in general, and the small scale of the data generated as well as the lack of field experience on the behavior of the treated soils in particular.

Due to the urgent need to stabilize eastern Saudi marls for construction purposes, this research program has been initiated. The following are the primary objectives of this investigation:

- i) To characterize the marl soils obtained from different areas in the Eastern Province of Saudi Arabia. The main task was to completely define their behavior, and point out the problems when these soils are to be used as a construction material.
- ii) To carry out detailed characterization and stabilization of two typical marl types, having poor behavior based on the evaluation in part (i). Different stabilization techniques were used to improve the performance of these soils for different construction purposes.
- iii) To evaluate the performance of the stabilized marls taking into account the effect of different parameters on the strength and durability. The parameters included different curing regimes, curing temperature, curing time, delay in compaction, and wetting and drying. The effects of these parameters on strength and durability were investigated using unconfined compressive strength (UCS), resilient modulus (M_R), and durability (ASTM D 559 and

Modified Slake Durability) tests.

- v) To quantify the effect of grain crushing due to compaction for one of the marl soil by using different laboratory compaction techniques.

Chapter 2

Literature Review

Calcareous soils, found in many parts of the world, are not normal soils. Engineers dealing with these soils are facing many problems including variability of their constituents, extreme water sensitivity, crushing of particles, and inapplicability of standard tests and specifications. In some areas, certain tests, specifications and stabilization techniques have been proposed for these soils. This chapter is planned to provide a background on the geological formation of these soils, and their behavior as observed by the engineers and researchers dealing with them.

2.1 Formation of Calcareous Soils

The basic constituent of calcareous soils is calcium carbonate. Calcium is furnished by the silicate minerals such as plagioclase feldspar, hornblende, and amphiboles. The chemical reaction of calcium with atmospheric carbon dioxide results in the formation of inorganic calcium carbonate, known as the calcite mineral (Garg, 1991). The contribution of calcite in the formation of calcareous soils is either in the form of clastic grains or in the form of solute in water. The other main origin of calcium carbonate is organic. The skeleton and hard parts of the bodies of marine organisms are calcareous in nature. They settle at the bottom of sea on the demise

of these organisms. Therefore, calcareous rocks exist only as sedimentary rocks. Later on, these rocks may be metamorphosed. The sedimentation process takes place in large water bodies. The sedimentary calcareous soils formed in oceans and lakes are termed marine and lacustrine deposits, respectively. These become terrigenous on the regression of sea. Chemical and mechanical weathering of these rocks causes the formation of sedimentary calcareous soils. The soils containing calcareous materials that are derived from a parent calcareous rock either by wind or flowing water are called non-sedimentary calcareous soils.

Calcareous material is susceptible to dissolution in acidic waters. Upon contact with atmospheric carbon dioxide, dissolution will occur and carbonic acid (H_2CO_3) will form. The extent of dissolution depends on the quantity of carbon dioxide dissolved in water, which in turn depends on the partial pressure of carbon dioxide and pH of water. The air in soil voids and therefore the interstitial water usually has partial pressures of carbon dioxide as high as 10 to 100 times its pressure in the free atmosphere (Horta, 1988). This results in high dissolution rate of carbonates in the interstitial water present. Water carries the dissolved calcareous materials from the parent rock to the host soil.

The precipitation of calcareous materials can also take place from the ground water containing dissolved carbonates. This results from the decrease in the partial pressure of the dissolved carbon dioxide. The reduction in partial pressure is initiated by the suction gradients produced by the transpiration loss through plants (Horta, 1988) and by the capillary rise. Carbon dioxide escapes from water as a result of the negative pressures in the capillary fringe. High transpiration loss is

enhanced in arid or semi-arid climatic conditions, in which evapotranspiration rate exceeds the precipitation rate. Consequently, the dissolved material significantly precipitates in the host soil in arid and semi-arid areas.








2.1.1 Sedimentary Calcareous Soils

The sedimentary calcareous rocks are formed by the cyclic precipitation, deposition and consolidation of the dissolved and suspended CaCO_3 in water. The sediments are either formed from detritus or chemical and biochemical calcium carbonate particles. The detritus grains are also termed "clastic" grains. The geological origin of the calcium carbonate can be one of the following (Bringen et al., 1982):

1. Clastic or grains of inorganic origin.
2. Bioclastic or grains of organic origin.
3. Oolitic or spherical grains coated with precipitated carbonate.
4. Pelitic or elliptical grains coated with precipitated calcium carbonate.
5. Reefoidal or grains consisting of old reef fragments.

The clastic calcium carbonate is derived from crystals of calcite and aragonite, amorphous calcium carbonate and hard parts of the organisms (Blyth and de Freitas, 1985). The bioclastic "organic" sedimentary carbonate deposits are found in one of six forms (Chaney et al., 1982) : (1) foraminiferan shells, (2) pteropod shells, (3) coccolithic plants, (4) corals, (5) precipitates, and (6) benthic materials. Table 2.1 summarizes information about marine carbonate minerals in terms of size and shape of aggregate particles, shape of sub-particles, depth location, environment, porosity, overall material behavior, and chemical constituent. These

Table 2.1: Marine Carbonate Materials (from Demars, 1982)

Name	Shape	Smallest Subparticle Shape	Depth Location, m	Environment		Porosity	Overall Material Behavior*	Chemical Constituent
				High-Energy area	Low-Energy area			
Foraminiferan shells (tests)			3500	benthic forams	planktonic forams	porous (porcellaneous)	cohesionless (sandy-silt)	calcite (Mg-calcite in small quantities)
Pteropod shells (tests)		no discrete subparticles	3000	tropical and subtropical waters	globigerina	porous (porcellaneous)	cohesionless (sand)	aragonite
Coccolithic plants (nanno-fossils)			3500 to 5000	---	mainly latitudinal variation of species type	no inherent porosity (hyaline)	cohesionless (silt)	calcite shells of the algal family - skeletons may be protected by organic or MgCO ₃ coatings
Corals		no discrete subparticles	0 to 35	branching, massive and encrusting corals	foliose and encrusting corals	porous (porcellaneous)	cohesionless (sand-gravel)	Mg-calcite aragonite
Precipitate	 highly variable	no discrete subparticles	35 to 200	oolites	pelletoids	no inherent porosity (hyaline)	cohesionless (sand)	Mg-calcite
Benthic materials	---	---	35 to 200	branching coralline algae, forams, planktonic feeders	mollusks, benthic forams, echinoid fragments	---	---	Mg-calcite

* Qualitative description of material response (that is, sand, sandy silt, sandy-gravel, silt) based on assumed particle size.

particles cover the full range of soil sizes from gravel-sized coral (and shell) fragments to sand-sized foraminifera, pteropod shells, and oolites to fine silt and clay-sized nanno-fossils. These organisms are found in the surface waters of the sea. Oolitic carbonate particles are rounded grains formed by the deposition of successive coats of calcium carbonate around particles such as a grain of sand or piece of shell (Blyth and de Freitas, 1985). Pelitic particles are similar to oolitic ones except for their elliptical shape. Reefoidal limestone develops from reef, a state resulting chiefly from the growth of an organically-constructed framework, lying at or just above or below the surface of water (Challinor, 1978).

During the sedimentation process, the soluble carbonate ions enter the sediment either by direct precipitation when the water becomes saturated because of evaporation or by the secretion (in the form of the shells and the bodies) of calcium carbonate (CaCO_3) caused by living organisms (i.e. aquatic fauna and flora) to develop a hard part or a skeleton which eventually comprises inorganic sedimentary particles (Demars and Chaney, 1982). The composition and textures of sedimentary rocks are controlled by the processes during their formation (Blyth and de Freitas, 1985). These processes are in turn controlled by the environment in which sedimentation takes place. There are three main environments namely continental, shelf sea and open or deep sea.

The continental environments prevail on land areas and include eolian, piedmont, alluvial, lacustrine and glacial deposits. These are respectively formed in deserts, foot of the mountains, rivers, lakes and glaciers. On the other hand, the shelf sea environment constitutes the deposition of calcareous material, muds, peb-

bles and sands at the margin of a sea on the continental shelf. In this case, the calcareous material is derived from the calcareous skeletal remains. The formation of different strata depends on the transgression and regression of the sea towards and away from the land. During transgression, the early deposited pebbles and sands are covered by finer sediments. These include fine sandstone and deep sea marine shale. During the retreat of the sea, the sediments deposited are different from that deposited during transgression. These are lagoonal deposits in the form of marine sandstone, shale, marl, eolian sand, and saline deposits. The sequences of the deposition of strata are shown in Fig. 2.1. The deep sea environment is characterized by the presence of calcareous and siliceous sediments derived from the skeletons of minute floating organisms and absence of large fragments. Clay deposits are found at greater depths where there are no surface current and wave action.

The sedimentary process for the carbonate rocks can be divided into two parts; one which occurred anciently and the other is currently occurring. The rocks formed by ancient sedimentary processes may sometimes be far removed from the present day sedimentary environments mainly by the regression of the sea from the land or the rise of the sea bed. Examples include the Arabian Gulf coastal plains, the Alabama black belt soils, the Pierre shale of North Dakota (USA) and even the top of the Mount Everest (Garg, 1991). Mount Everest is an example of the upheaval of sedimentary deposits which were originally formed under the sea.

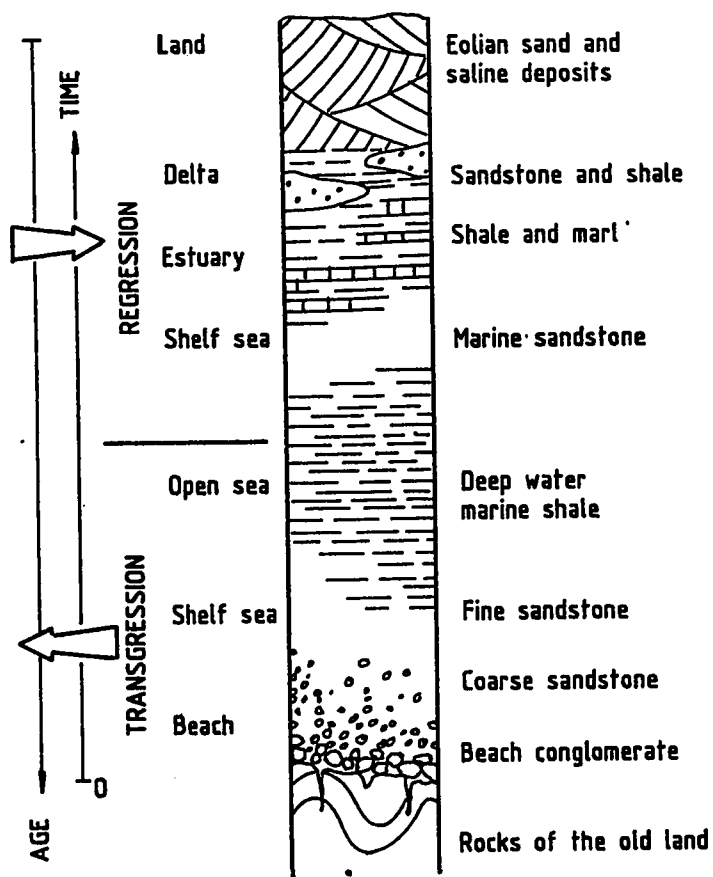


Figure 2.1: Possible Succession of Rocks Formed During Transgression or Regression of the Sea (from Blyth and deFreitas, 1985) No scale implied.

Limestone, Dolomite, and Marl

The precipitation, deposition and consolidation of calcium carbonate results in the formation of limestone, while that of clay results in mudstone or shale. Calcareous materials deposit in high energy areas while carbonate muds deposit in quiet lagoons (Al-Sayari and Zotl, 1978). The high energy area is characterized by high turbulence of sea water, keeping the fine clay particles to remain in suspension. While, in the low energy areas, the clay and other sediments settle simultaneously to provide the dual character to the rock. After deposition, limestone undergoes mineralogical changes to form dolomite, a process known as dolomitization.

The rock formed due to simultaneous deposition of calcareous material and clay is termed marl. There exists large differences in marl types and characteristics, therefore, there is no unanimous consensus on the proper definition for marl. However, some of the definitions used by different authors are given in Table 2.2. The outcome of the different definitions used is that the marl is a mixture of calcium carbonate and clay.

2.1.2 Sedimentary Calcareous Rocks in the Arabian Peninsula

The Arabian Peninsula is partly covered by calcareous rocks and soils. Lying between Africa and Western Asia, the Peninsula can be divided into two structural provinces, namely the Arabian Shield which is located on the west and the Arabian Shelf towards the east (Al-Sayari and Zotl, 1978). Generally, the Arabian Shield is part of the Precambrian crustal plate, locally covered by Tertiary volcanic rocks

Table 2.2: Marl Definitions and Attributes Used by Different Authors

Author(s)	Year	Definitions and Attributes
Terzaghi and Peck	1967	Stiff to very stiff marine calcareous clays of greenish color.
Pettijohn	1975	Soil or rock like material containing 35-65% carbonate and a complementary content of clay.
Fookes and Higginbottom	1975	A simple binary mixture of calcium carbonate and clay.
Mitchell J. K.	1976 1993	Marl is ranging from relatively pure calcium carbonate to a mixture of calcium carbonate with mud and organic matter formed by biochemical processes.
McCarthy	1977	A soft limestone
Challinor	1978	A mixed rock containing clay minerals and aragonite or calcite, usually together with accessory components, such as silt, in lesser quantity.
Saudi-ARAMCO	1978	Soft limestone contaminated with varying amounts of clay
Sowers and Sowers	1979	Water-deposited sand, silt or clay containing calcium carbonate
Bates and Jackson	1980	It is an old term that is generally loosely applied to a variety of materials most of which consist of an intimate mixture of clay and calcium carbonate.
Mitchell R. S.	1985	Soft calcareous clay-rich mineral
Blyth and de Freitas	1985	Calcareous mudstone
Al-Tayyib et al.	1985	Carbonate soils, the formation of which is attributed to physical and chemical weathering of parent carbonate rocks
McLean and Gribble	1985	Friable carbonate earths deposited in freshwater lakes
McLean and Gribble	1985	Fig. 2.2.
Qahwash	1989	Calcareous sediments
Aiban	1994	Fine-grained calcareous sediments

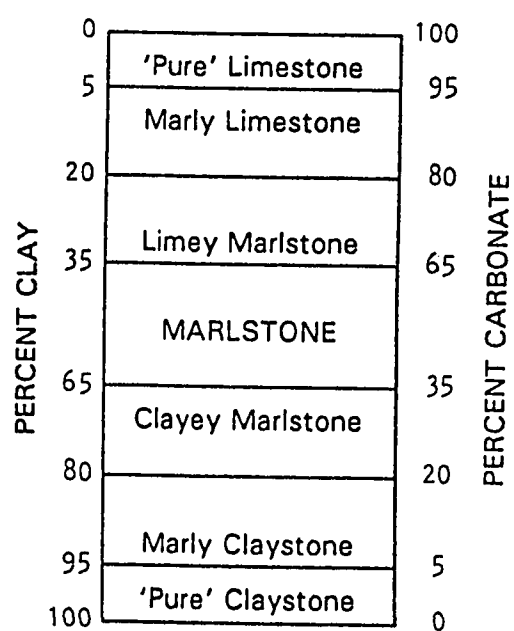


Figure 2.2: Geological Classification of Marls (from McLean and Gribble, 1985).

at some places. In the Arabian Shelf, the plate is covered by thick sedimentary sequences. The Eastern Province of Saudi Arabia is a part of the Arabian Shelf. The calcareous rocks and soils in the area is a result of millions of years old sedimentation processes which prevailed in the area. The discussion will be restricted to the formation of calcareous rocks during different geological periods within the Arabian Shelf.

The Arabian Peninsula is composed of Precambrian sedimentary and volcanic rocks that was metamorphosed thereafter and have intruded plutonic formations. At that time, the Arabian Peninsula was attached to the African Shield (Al-Sayari and Zotl, 1978). They stated that at the start of Cambrian Age, a great sedimentary basin developed north and east of Arabia. Throughout Paleozoic, Mesozoic, and early Cenozoic times, many thousands of meters of sediments accumulated in this deep, slowly-sinking trough (Al-Sayari and Zotl, 1978). The sedimentation basin was the old Mediterranean, known as the Tethys. Between the Tethys and the Arabian Peninsula, there were broad epicontinental seas. These were responsible for depositing Paleozoic, Mesozoic, and early Cenozoic strata over the eastern part of the Peninsula. Continental sedimentation was simultaneously taking place. The continental sediments were overlain by marine deposits due to the back and forth movements of the shoreline. The shoreline movement was caused by the slight vertical movement of the crust. The Arabian Plate separated from the African Shield in the middle Tertiary age whereby the Red Sea acted as a border (Al-Sayari and Zotl, 1978). The sediments deposited at different ages were not disturbed by different geological changes such as the formation of mountains, separation and movement of crustal plates.

The eastern edge of the Arabian Peninsula, from the southern end of the Arabian Gulf northwestward through Iraq and into Syria, lies in a rectangular depression formerly known as Mesopotamia (Al-Sayari and Zotl, 1978). The southern part of the depression includes the western half of the Arabian Gulf and a narrow strip of the Arabian Peninsula, known as the Arabian Gulf coastal region. The surface rocks of this part of Saudi Arabia include consolidated sedimentary formations ranging from Paleocene to Middle Eocene age and Miocene to Pliocene age. Unconsolidated materials of Tertiary age and sediments of Quaternary age are also present. The surface formations of the region are shown in Table 2.3. and Fig. 2.3.

The primary sedimentary rock formed from the calcium carbonate sedimentation is limestone. The limestone can be subsequently dolomitized and therefore converted to dolomite. Besides calcite and dolomite, accessory minerals such as flint, chert, and jasper are also formed. These result from a process called *metasomatism*. Metasomatism (a Greek word meaning "change in substance") involves the removal of chemical constituents or the introduction of new ones, resulting in chemical subtraction, addition, or replacement (Challinor, 1978). It is most commonly used for the production of new minerals by the introduction of new elements from igneous sources or by percolating solutions. These processes are responsible for the formation of dolomite, chert, and other materials in calcareous soils.

Dolomite or dolomitic limestone are rocks which contain the double carbonate, $\text{MgCO}_3 \cdot \text{CaCO}_3$ (Blythe and deFreitas, 1985). While the magnesian limestone (i.e.,

Table 2.3 : Geological Formations of Eastern Saudi Arabia (after Roger, 1985)

Formation	Lithology
Hofuf	CONGLOMERATE: Red and white conglomerate; boulders and pebbles of limestone in quartz matrix. Basalt unit greenish-gray to red, in part sandy marl.
Dam	MARL, CLAY, and LIMESTONE: Upper part red to green partly sandy and silty clay with minor marl and sandstone beds and buff to white limestone with yellow, fossiliferous limestone and sand; lower part white very calcareous fossiliferous marl; minor sand and clay.
Hadrukh	SHALE, SANDSTONE, and MARL: Green and grayish-green generally finely sandy clay and green to gray calcareous sandstone; commonly weathers to fine concretionary pellets. Minor amounts of cream to gray marl and gypsum are also present. Chert occurs at a number of levels. Basal 5-10 m commonly cream sandy limestone or marl.
Dammam	<u>Alat</u> : LIMESTONE and MARL: Upper part light-colored chalky porous commonly dolomitic limestone; abundant molds of mollusks and other organic remains. Lower part light-colored dolomitic marl. <u>Khobar</u> : LIMESTONE and MARL: Light- to dark-brown, in part dolomitic limestone becoming off-white soft marly limestone and marl in lower part. <u>Alveolina Limestone</u> : LIMESTONE: Tan limestone. <u>Shaila Shale</u> : SHALE: Poorly exposed yellow-brown shale.
Rus	LIMESTONE: White soft chalky porous limestone; calcarenite beds at top. MARL and LIMESTONE: Light-colored marl; local irregular masses of crystalline gypsum, occasional thin limestone beds and geodal quartz at several levels. In other areas, unit is highly variable, including as common equivalents: (a) white compact finely crystalline anhydrite with interbedded green shale and minor amounts of dolomitic limestone or (b) gray marl with coarsely crystalline calcite and interbedded shale and limestone. LIMESTONE: Gray to buff compact commonly partially dolomitized limestone; minor amounts of soft limestone made porous by leaching of small organic remains. Quartz geodes occur rarely in the lower part, and are typical of upper part.
Umm er Radhuma	DOLOMITE: contains <i>Lockhartia tipperi</i> Davies.

LEGEND

	Eolian Sand (including Sabkha deposits)		Dam Fm.
	Dammam Fm.		Hadruk Fm.
	Neogene und.		Umm Er Radhuma Fm.
	Hofuf Fm.		Rus Fm.

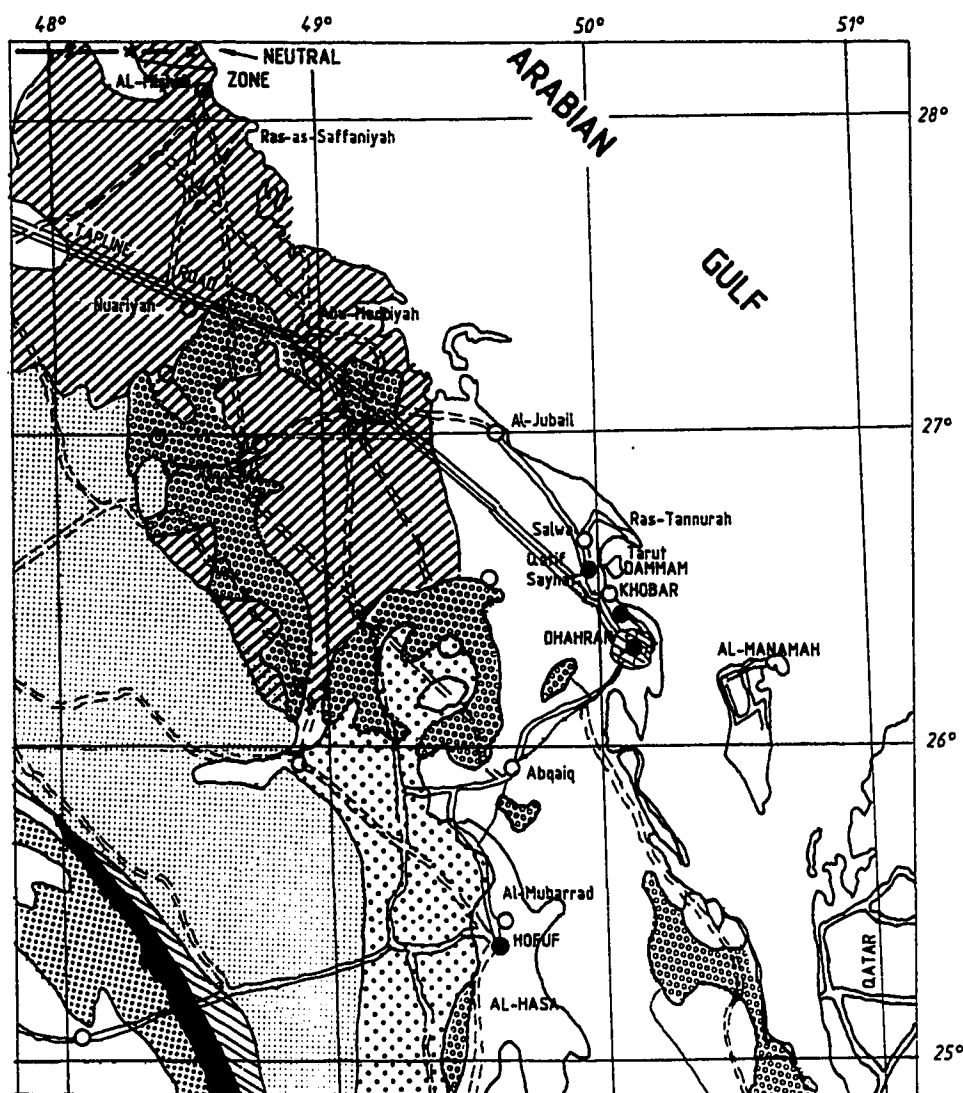


Figure 2.3: Geological Formations of Eastern Saudi Arabia (After Ministry of Agriculture and Water, 1969).

magnesite) contains magnesium carbonate alone, it does not exist as double carbonate as it is held in a solid solution in calcite crystals. Sea water contains Mg^{2+} and SO_4^{2-} in solution, and when it permeates through limestone rock, the magnesium replaces calcium in the calcite crystal resulting in the formation of dolomite mineral. The process is termed "dolomitization". The calcium ion released from the crystal lattice of the calcite combines with sulphate ion and precipitates as nodules of anhydrite ($CaSO_4$) or gypsum ($CaSO_4 \cdot 2H_2O$). The change of crystal structure from calcite to dolomite results in a volume reduction of 12.3% and hence often results in a porous texture (Blythe and deFreitas, 1985). The change usually takes place in shallow waters, whereby dolomitization of the recently deposited aragonite mud and simultaneous formation of anhydrite and gypsum occurs in the tidal zones of sabkha.

Formation of silicified limestone and chert is another form of metasomatism. It results from the replacement of calcium carbonate by silica. Silica, from organisms such as sponges and radiolaria, is dissolved by water containing potassium carbonate, and re-deposited as chert, which is a form of cryptocrystalline silica (Blythe and deFreitas, 1985). Deposits of sponge spicules, which are fragments of the skeletons of siliceous sponges, are found today on parts of the ocean floor. Older deposits of this kind provided the raw material for the cherts which occur in certain limestone. Flint and jasper are different forms of chert found in chalk and limestone beds. Replacement of calcite by siderite ($FeCO_3$) is another important metasomatic change (Blythe and deFreitas, 1985).

2.1.3 Non-Sedimentary Calcareous Soils

The non-sedimentary calcareous soils are formed by the precipitation of calcareous material in a host soil. Some of this material is derived from the parent calcareous soil by physical or chemical weathering while others are derived from an influx of carbonate-rich aeolian dust or carbonates dissolved in rainwater (Demars and Chaney, 1982). During intermittent periods of wetting in arid or semi-arid climates, the dissolved carbonates penetrate the parent materials and then precipitate by evapotranspiration processes. Precipitation occurs at the grain contacts as a result of the capillary action. Netterberg (1982) noted that calcretes may form when dissolved carbonates precipitate from the ground or soil water. In any case, the formation of caliche or calcrete soils involves complex geochemical and geophysical processes which are dependent upon local geological and environmental conditions. The particle size for these non-sedimentary carbonates varies from a silt-sized powder to cemented nodules to hardpan or massive rock. The host material for non-sedimentary carbonates can be of clay, silt or sand size (Horta, 1988).

Calcrete

The term "calcrete" came from the Latin words "calx" meaning lime and "crescere" meaning grow. The term "calcrete" is used in southern Africa and is usually applied somewhat loosely to any soil cemented or replaced by calcium carbonate of any origin (Netterberg, 1967). Calcretes (also called caliches, as it is known in USA) are common in arid and semi-arid lands, and probably rated as the most widely used road construction material in South Africa (Netterberg, 1982). "Calcrete" can be defined as a material formed by the *in-situ* cementation or

replacement or both of a pre-existing soil by carbonates, precipitated from the groundwater or soil water (Netterberg, 1982). The term "soil" is used here in its wide engineering sense, for any material which does not require blasting for excavation. Netterberg (1982) stated that calcretes vary widely in carbonate content and geotechnical properties from loose, fine-grained soils to limestone rock, and it is this variability which has been a major cause of confusion and disagreement over their geotechnical properties and behavior.

Calcrete materials represent a mixture of different types of elements with different sizes including hardpan cobbles, crust fragments, nodules, calcrete fines, and various types of host material and inherited elements such as pebbles, eolian sands, silts and clays. Calcrete fines mainly comprise of calcareous silt, fine crystals of calcite (micrite) and a clay fraction. The clay fraction may be inherited from the host material and/or formed during the calcrete development process. Studies by many authors in several parts of the world concluded that the neoformation of attapulgite is characteristic of calcrete development (Horta, 1988). It was found that although attapulgite exhibits high plasticity indices, it is stable and does not swell. The inherited clay minerals may include practically all other clay minerals such as kaolinite, illite and the highly unstable montmorillonite (Horta, 1988).

In North Africa, calcrete is currently referred to as "tuf" (Horta, 1988). This term seems to be related to the Arab word "tafezza" meaning soft sandstone and to the Barberian word "tifkert", synonymous with crust or tartar. The word "tuf" has been adopted by road engineers, although it was at once rejected by geologists and replaced by "croute" and "encroutement calcarie", which can be translated

into English as encrusting and calcareous crust respectively.

Caliche predominates most of Arizona, New Mexico, and in the adjacent parts of the southwestern United States and Mexico. These soils possess distinctive geotechnical properties that are quite different from those generally treated in classical soil mechanics. These properties are a consequence of their formation in arid and semi-arid periods during which calcium carbonates were carried downward by percolation of water from the surface, precipitating to form cemented horizons (Beckwith and Hansen, 1982). These soils possess highly "collapsing" characteristics. The calcareous soil horizons of Tucson Basin, USA, are usually found a few feet below the existing ground level, but may also exist at a considerable depth (Post, 1982). In some areas, the caliche soil horizons are a few feet thick and in others only a few inches thick. According to Post (1982), the caliche varies from a finely disseminated calcium carbonate in the soil to a dense indurated material of considerable strength.

Calcrete or Calcareous Duricrust in the Arabian Peninsula

Throughout the Shedgum area of the Eastern Province; the plateau surface, the tops of erosional outliers, the sides of wadis and gullies, and all terrace surfaces are covered by duricrust which has developed on the calcareous beds of the Hofuf Formation (Al-Sayari and Zotl, 1978). The events responsible for the formation of duricrust started with uplift of the plateau. Later on, during the semi-arid periods, the duricrust resulted from the recrystallization of original calcite in the parent sandy limestone, by the introduction of additional calcite either from above or from below; the additional calcite replaced quartz and feldspar and removed

the released silica (Al-Sayari and Zotl, 1978).

2.2 Geotechnical Properties of Calcareous Soils

2.2.1 Atterberg Limits

Calcareous soils are formed by the weathering of rocks containing calcium carbonate as their main constituent. The source of calcium carbonate constituting these rocks is mostly organic. Organic carbonates are skeletal remains of the microscopic plants and animals settled at the sea floor such as pteropods, foraminifera (forams), and coccolithophorids (nannofossils or nannos because of their small size) (Demars and Chaney, 1982). These organisms live in the surface waters of the sea; and, on demise, they settle to the sea floor.

The shells or skeletons of these organisms are porous in nature, containing intraparticle water. Demars and Chaney (1982) estimated that the ratio of the volume of intraparticle water to the volume of particle solids to be 5.0 for forams and 1.05 for nannos. This intraparticle water is responsible for the erroneous values of the liquid and plastic limits (LL and PL) of the carbonate soils. Atterberg limit tests are frequently performed on the soil fraction passing ASTM No. 40 sieve (ASTM D423, ASTM D424). The water contributing towards the plasticity behavior of the soil particles is only the surface or adsorbed water portion of the interparticle water. The moisture content determination for a soil by drying the sample is the sum of the interparticle and intraparticle water. The apparent liquid and plastic limits determined for a calcareous soil are always higher than their actual values. Since the difference of the two values eliminates the above-mentioned effect, the plasticity index (PI) is therefore not greatly affected by the presence

of intraparticle water (Demars, 1982). In addition, the shrinkage limit (SL) for calcretes is higher than that for normal soils ($SL > 20\%$) and is found to be very close to or even greater than the plastic limit (Horta, 1980).

Calcium carbonate equivalent (carbonate content of the material passing the ASTM No. 40 sieve) of calcareous soils can be used in place of PI for the construction material (Horta, 1988). It was found that PI decreases with an increase in calcium carbonate content. The determination of the calcium carbonate content is carried out in a COLLIN-BERNARD calcimeter or in a carbonate bomb. It is a direct measure of the pressure of the carbon dioxide released during the reaction of calcium carbonate with hydrochloric acid. The test is simple and less time consuming compared to Atterberg limits tests.

Field Pliers Test is used for calcrete gravel roads instead of PI in South Africa (Netterberg, 1967). This test is simple and can be used to determine the hardness of the aggregate. The method of test involves obtaining 100 to 200 pieces of the air dry $1/2$ in. (12.7 mm) to $3/4$ in. (19.0 mm) aggregate portion of the nodular calcrete gravel by sieving or hand picking and firstly trying to break the pieces between the thumbs and forefingers, using both hands at once. The unbreakable pieces are then tested with a standard pair of pliers (Hazet 1850- $3/7$ in.) in the concave serrated portion of the jaws, using only one hand. The total percentage passing fingers and pliers are respectively designated as Aggregate Fingers Value (A.F.V.) and Aggregate Pliers Value (A.P.V.), which are a measure of the hardness of the aggregate. The hardness of calcrete gravel fraction is found to be more important than the PI of the fine fraction.

The specified maximum accepted value of PI for the soil to be used for the construction of base course is 6%. However, for calcretes, a higher value of PI than 6% can safely be used (Horta, 1988). The use of the above specified limit can result in the discard of good construction calcareous soils. The upper limit of PI is therefore relaxed and values upto 10 to 15% can be accepted for the use in base course construction.

Akili (1980) found that the behavior of marl, obtained from the south of Dammam city, eastern Saudi Arabia, was affected by the presence of plastic material. The plasticity in the marl was due the presence of clay. Similarly, Qahwash (1989) found that the liquid limits and PI's of the calcareous sediments in the Dammam-Dhahran area, were attributed to the existence of some montmorillonite and/or soluble salts as displayed by his X-Ray diffraction (XRD) results.

Tsiambaos (1991), while working on the marls from the city of Iraklion (Greece), found large variations in the Atterberg limits of these soils. One of the group of Iraklion marl exhibits LL values in the range of 24 to 59% and PI of 3.5 to 36% while another group showed a variation in LL from 33 to 100% and PI values from 10 to 57%.

2.2.2 Grain-Size Distribution

The grain-size distribution of non-sedimentary calcareous soils depends on the host soil which varies from clay to gravel size. The sedimentary calcareous rocks are formed by the deposition and the consequent cementation and induration of

the calcium carbonate particles in the marine environment. The cementation is mostly due the calcite precipitation. These calcium carbonate particles are mostly of biological origin as in Table 2-1. The grain-size distribution of the calcareous soils depends on the degree of cementation of the soil grains (Datta et al., 1982). The cemented particles can lead to misleading grain-size analysis because the cementation is lost upon remolding and to some extent by the dissolution in water. This dissolution is pronounced if the water is acidic ($\text{pH} < 7$).

The grain-size distribution of calcareous soils is not representative of their engineering behavior if cementation and/or induration is not considered in the analysis (Datta et al., 1982). In the case of cemented calcareous soils, the specific gravity of the fines is higher than the coarser particles formed by the cementation of the finer particles whereby the calcium carbonate is the main cementing agent. The grain size distribution by weight of calcareous soils, such as calcrete, overestimates the volume occupied by the fine particles. In addition, the washed and dry sieve analysis show large differences in the percentage of fines, the unwashed sieving being unreliable for calcareous soils. Moreover, when the fines (passing ASTM No. 200 sieve) are subjected to the hydrometer analysis for estimating their grain-size distribution, the hollow particles float on the surface of the water, making the test unreliable for these types of soils (Datta et al., 1982).

Tsiambaos (1991) showed a great heterogeneous nature of Iraklion (Greece) marls. He reported that the members of one group of marls contain sand (2-34%), silt (45-62%) and clay (10-45%), while another group contains a lower percentage of sand (4-14%) and clay (4-38%) but a higher percentage of silt (65-90%).

2.2.3 Strength

Strength of soil is function of its structure, which in turn depends on the particle orientation and their interaction with each other. Strength of calcareous soils is controlled by some additional factors such as grain crushing, dissolution in pore water, etc. The grain crushing results in reduction in the angle of internal friction at relatively low stress levels. Further, the sand-sized carbonate particles offer lesser frictional resistance as compared to quartz sand grains. The strength of calcareous soils is also controlled by the carbonate cementation at the grain contacts. Carbonate-cemented soils show higher strengths, but dissolution of this cementation in pore water results in a complete loss of cohesion.

Some areas of Southwestern United States, such as Arizona, New Mexico and the adjacent areas, are covered with non-sedimentary calcareous soils. These are formed by the precipitation of dissolved calcium carbonate from the parent rock, brought down by the flowing water (Beckwith and Hansen, 1982). The precipitation of calcium carbonate provides different degrees of cementation. Depending on the degree of cementation, the authors classified these soils into 5 classes. Class 1 soils are the most weakly cemented soil and while Class 5 soil is moderately hard rock. The bearing capacity of shallow foundations on Class 1 to 3 soils is usually evaluated by the Terzaghi's bearing capacity equation or similar ones. Class 1 soils are very much moisture sensitive, and therefore, are classified as 'collapsible' soils. For Class 4 and 5, the shear strength is evaluated by an approach which employs rock mass classification system. Beckwith and Hansen (1982) found that the 'adhesion' to be used for the design of drilled piers in these soils (Class 2 to 5) is approximately equal to their undrained shear strength. The side friction of

drilled piers mobilized in calcareous soils is much higher than the range for stiff, fissured clays.

Valent et al. (1982) evaluated the geotechnical properties of two calcareous oozes. They defined calcareous oozes as the clay to fine-sand sized remains of calcareous marine life, covering approximately 36% of the deep-sea floor. In geology, oozes are pelagic deposits pertaining to the open ocean and composed entirely or predominantly of the hard parts of small planktonic organisms (Challinor, 1978). Valent et al. (1982) obtained these oozes from the Venezuelan Basin at a water depth of 3930 m. One of the ooze, labeled as clayey silt (56-75% calcium carbonate), was subjected to isotropically-consolidated undrained (CIU) triaxial shear strength test. It was observed that the angle of internal friction was reduced from 34° to 28° when the confining pressure was raised above 4 psi (27.6 kPa). Another ooze, designated as fine sand-silt, showed a similar behavior of reduction of the angle of internal friction. They attributed this behavior to the probable crushing of soft, calcareous grains at high confining stresses.

Datta et al. (1982) studied the carbonate soils of the Indian continental shelf. They studied both the surface sediments as well as the subsurface deposits. They referred to these soils as carbonate soils since they contained both calcium carbonate and "dolomitic" (i.e., calcium-magnesium carbonate) contents. These soils were subjected to isotropic compression tests and drained triaxial shear tests under confining pressures of 1 to 64 kg/cm² (98.1 to 6278.4 kPa). The degree of crushing of the soil grains was evaluated by comparing the grain-size distribution curves before and after the test. A crushing coefficient was defined as the ratio

of the percentage of the particles finer than D_{10} of the soil after being subjected to stress to that of the original soil. Crushing coefficient values increased with increasing the confining stress. The Value of crushing coefficient at a confining stress of 64 kg/cm^2 (6288.2 kPa) was defined as the susceptibility of crushing (S_c). They found that crushing increases with an increase in: (1) the amount of grains having large intraparticle voids, (2) the amount of thin-walled shell fragments, (3) the angularity of grains, (4) the coarseness of grains, and (5) the uniformity of gradation. Crushing was found to be independent of the carbonate content of the soil. The crushing of grains caused the drained angle of shearing resistance to decrease. Higher crushing coefficient caused a greater reduction in the angle of shearing resistance. They also found that the cementation of fine-grained carbonate soils make them behaving like coarse-grained soils.

Bringen et al. (1982) evaluated the grain crushing of carbonate soils during consolidation and shearing tests. They compared carbonate sands obtained from offshore Bombay, India and North Rankin site, offshore Western Australia, with a conventional quartz sand from Maas river in Holland and Belgium. It was found that the amount of crushing was larger for soils which have larger, uniform, or angular particles. From triaxial tests, it was found that the crushing increases with both consolidation pressure and deviator stress. The crushing coefficient, as defined by Datta et al. (1982), was found to be an appropriate parameter for the evaluation of the crushing behavior of soil grains.

Post (1982) investigated the caliche soils found in the Tucson area, Arizona, USA. Unconfined Compressive strength of caliche was performed on both natu-

ral indurated caliche and a remolded one. For the indurated caliche, blocks for compressive strength testing were cut with a masonry saw. On the other hand, bulk samples were reduced to material passing ASTM No. 20 sieve and was used for remolded soil. The remolded soil was extremely moisture sensitive. A complete collapse of bearing strength occurred upon saturation. Presence of clay in the case of indurated samples weakens them, while in the case of remolded soil it was responsible for the cohesion of the soil particles. This cohesion is lost upon saturation and collapse occurs.

The playa lake deposits in southern Idaho, USA, was studied by Olsen and Leonard (1982). The parent materials for these soils consist of sedimentary limestone and calcium rich basalts. These deposits contain 20-25% calcite. The authors reported the unconfined compressive strength results of remolded specimens. Prior to testing, the samples were cured for different time intervals ranging from several minutes to 3 months. These samples showed considerable strength gain (400%) with loss of moisture. The specimens compacted at the optimum moisture content showed higher strength compared to those at the dry side of optimum. Density and calcite cementation were the contributing factors for such behavior.

Arkin and Michaeli (1989) studied the strength and cementation of chalk and clay mixtures. The natural sediments were mixed in varying proportions. The mixtures were of the Upper Cretaceous smectites (bentonite and montmorillonite clay) and Jurassic kaolinite from Makhtesh Ramon with Eocene chalk from the northern Negev. These materials were obtained from different quarries located in the southern occupied Palestine. They tried to simulate the behavior of marl

rock within carbonate sequences, which occurs as varying proportions of clay and carbonate. Various proportions of the dry weight of the mixtures were prepared such that the carbonate content varied from 10 to 90 %. The effective range in friction angle for the kaolinite-chalk mix (KC) was $\phi=13-36^\circ$ with a corresponding range in cohesion $c=14-37$ kPa. The montmorillonite-chalk mix (MC) showed a friction angle range of $\phi=2-37^\circ$ and a cohesion of $c=11-36$ kPa, while that of bentonite-chalk mix (BC) showed $\phi=3-35^\circ$ and $c=18-40$ kN/m². The variations in the shear strength parameter values does not depend only on the presence of specific clay mineral, but also on the variation in carbonate content as the plastic limit exceeds 30. They attributed the variation in the strength to the lime-mud sediments undergoing lithogenesis. As the lithogenesis progresses, the properties of the sediment change accordingly.

Akili (1980) performed repeated loading tests on limestone and argillaceous limestone (marl) obtained from the south of Dammam city, eastern Saudi Arabia. The results show that for stress levels above a certain critical value, plastic strains increased with repeated stress application until failure of the sample occurred. Below the critical stress, plastic deformation continued to increase only with increased number of stress cycles even at very low stress levels. For both soils, the critical stress was approximately 40% of the undrained shear strength under static loading at the same static confining pressure. Increasing the number of stress cycles, led to an increase in the resilient modulus (impulse deviator stress divided by resilient or recoverable strain) of soil derived from limestone, and a decrease in the modulus of resilient of clayey soil. This indicates that the former is a strain hardening while the later is a strain softening material.

Tsiambaos (1991) developed correlation between the residual angle of friction for marl soils found in Greece and both calcium carbonate content and Atterberg limits. He found that any increase in the calcite content of these marls causes a decrease in their plasticity index and clay-sized fraction, but a significant increase in the residual angle of friction.

Aiban (1994) studied the strength of a marl obtained from Abqaiq, eastern Saudi Arabia. The results showed that the strength of samples compacted at optimum moisture content gave higher strength compared to the samples compacted on the dry as well as on the wet side of the optimum. Aiban (1995) explained the above behavior in relation to the fabric of the soil. On the dry side of optimum, the presence of small macropeds serves as a bridge or a connector between large macropeds. These macropeds are extremely sensitive to the moisture content and become weaker upon contact with water. This behavior is responsible for the lower strength on the wet side of optimum since the macropeds are too weak to resist the stresses imposed on them. At optimum, the macropeds are closely packed and have regular connectors.

2.2.4 Compressibility and Swelling

Calcareous soils consist of soft carbonate particles, which are susceptible to crushing when loaded or compacted. The loading generates high stresses at the particles' contacts. The carbonate particles are easily crushed even at low load levels, causing high compressibility. The compressibility of the calcareous soils is enhanced at stress levels initiating crushing of grains.

Valent et al. (1982) investigated the compressibility characteristics of two calcareous oozes. They performed consolidation tests on these samples at different load increment ratios. For each load increment ratio, samples were subjected to pressures ranging from 0.48 to 2490 kPa. They found that with a load increment ratio of 1, the curve of void ratio versus pressure (e -log p) showed no straight line portion even at high pressures. This typical behavior of calcareous soils is shown in Fig. 2.4. But with a load increment ratio of 2, a straight line portion was obtained as shown in Fig. 2.5. This behavior may have to do with the crushing of weak calcareous grains. However, no signs of grain crushing were found. Possibly, the grains underwent very high non-recoverable deformation of the hollow shells present in the soil. They found the average and median value of the Compression Index (C_c) to be 0.75 (average of values at two different stress increment ratio and median is the average of these averages obtained).

Compression index (C_c), as determined from empirical relations based on liquid limit (LL), water content, and initial void ratio were lower than the actual values. The reason for this is the low LL values of silty calcareous fines and their low water affinity. On the other hand, these may suffer high compressibility due to their fine-grained nature and crushing of the soft grains. C_c values based on moisture content were greater than the actual experimental values due to the presence of intraparticle water in the hollow calcareous particles. The intraparticle water does not contribute to the compression, although it appears in the formula for calculating the Compression Index. Valent et al. (1982) also studied the crushing of grains during consolidation tests. They found that the grain crushing took place

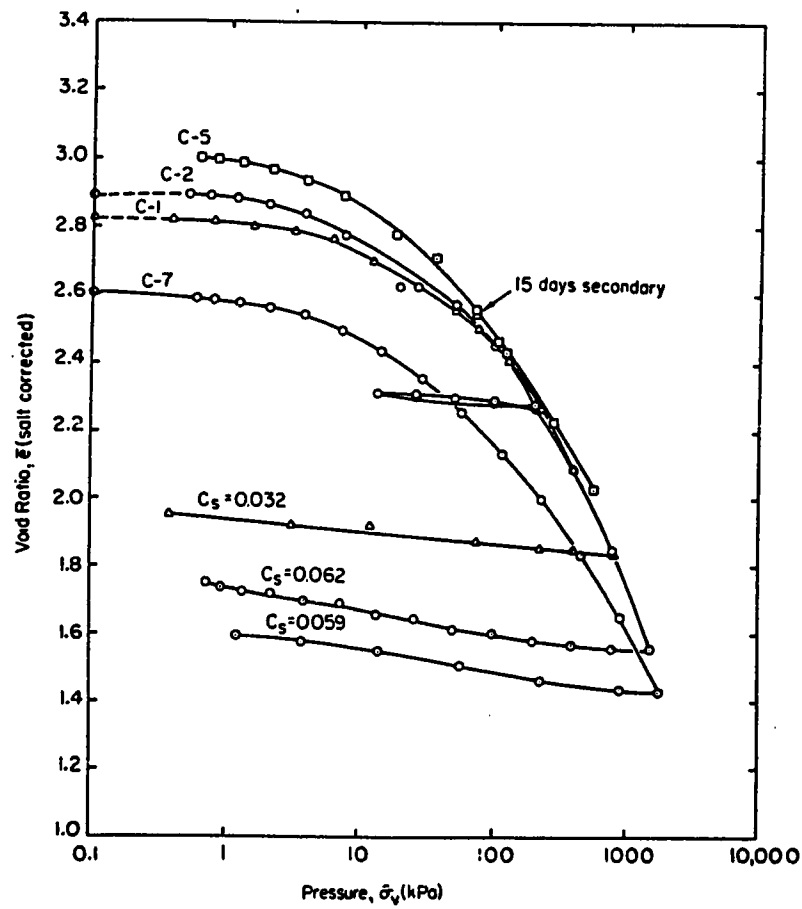


Figure 2.4: One-dimensional Compression of Natural Specimens, Load Increment Ratio = 1 (from Valent et al., 1982).

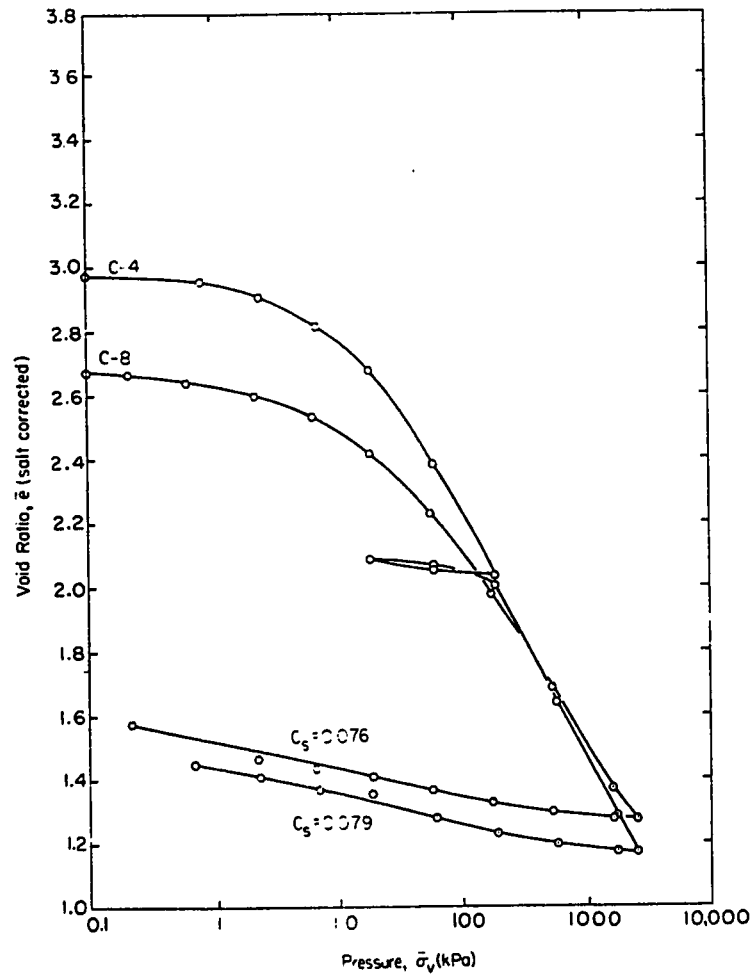


Figure 2.5: One-dimensional Compression of Natural Specimens, Load Increment Ratio = 2 (from Valent et al., 1982).

for the soil samples containing no fines. Crushing of calcareous grains takes place at 50 to 200 kPa, while for quartz sand it ranges from 14,000 to 21,000 kPa.

Demars (1982) investigated the compression behavior of calcareous clay cores obtained from the Eastern Atlantic, a few hundred miles off the coast of North Africa. The conventional laboratory oedometer was modified to permit falling head permeability measurements. The permeability was measured since it was thought that crushing of grains causes a marked decrease in void ratio and coefficient of permeability. Later, this hypothesis was found to be incorrect. He reached almost the same conclusions as that by Valent et al. (1982) where the coarse-grained particles crush easily if not surrounded by the fine material. The fine material acts as a cushion to the coarse particles. The crushing of calcareous particles, that have intraparticle voids, causes reduction in volume. This volume reduction is in addition to that caused by the decrease in void ratio.

McKown and Ladd (1982) studied the compressibility of Pierre shale. The Pierre shale formation underlies a large portion of the Great plains region of the north central United States. The formation contains CaCO_3 which also acts as the cementing material. They performed leaching/consolidation tests on samples obtained from unweathered Pierre formation. Consolidation tests were performed on three specimens: the first was in its natural state, the second was permeated with distilled water saturated with CaCO_3 , and the third was leached with HCl . They concluded that the natural cementation in cohesive soils shows an apparent past pressure in the consolidation test. This is caused by the cementation provided by a threshold value of about 2% CaCO_3 . An increase in percentage of CaCO_3

above this value (up to 50%) causes no further increase in cementation. Leaching with HCl caused a reduction in the CaCO_3 content of soil thereby increasing the compressibility of the soil. Permeation with distilled water saturated with CaCO_3 caused no change in behavior.

Arkin and Michaeli (1989) studied the free swell behavior of the chalk and clay mixtures. Various proportions of the dry weight of the mixtures were prepared such that the carbonate content varied from 10 to 90 %. The montmorillonite-chalk mix showed a rapid free swell within a relatively small change in carbonate content. The bentonite-chalk showed a relatively short range of free swell within a wide range of carbonate content, while kaolinite-chalk mix showed little variation with the change in carbonate content.

Aiban (1994) investigated the one-dimensional compressibility of a calcareous soil using the oedometer test. Samples were subjected to swelling and consolidation testing at optimum, on the wet side as well as on the dry side of optimum moisture contents. Samples compacted at optimum moisture content showed lower compressibility compared to samples compacted on the dry as well as on the wet side of optimum. On dry side, the samples showed higher compressibility when inundated with water.

2.3 Classification of Calcareous Soils

From an engineering point of view, a classification system is satisfactory if it can be achieved by performing simple tests, and if it provides a reasonable indication of the engineering behavior of the soil (Datta et al., 1982). For terrigenous

noncarbonate soils, the characteristics that have been found relevant for the classification purposes are particle size, particle-size distribution, and plasticity. It has been found that these characteristics are not sufficient for classification of carbonate soils because of the following:

- (i) Such classification systems would give no indication about either the susceptibility to crushing of carbonate particles or the nature of cementation in the soil.
- (ii) In soils with intraparticle voids, the Atterberg limits and grain-size distribution curves are not totally relevant because the water held in the intraparticle voids affects the behavior of these soils.
- (iii) In some of the fine-grained carbonate soils, it is not even possible to perform hydrometer analysis because hollow grains float on the surface of water or kept suspended in water, and moreover solubility of particles may be significant.

Therefore a modified or an entirely new system of classification is required for carbonate soils because carbonate soils significantly exhibit different engineering behavior compared to terrigenous non-carbonate soils.

2.3.1 Existing Classification Systems

Classification of carbonate sediments before the seventies was based on geological terminology. According to Datta et al. (1982), the basis of the terms used therein include:

- (i) Origin, for example, skeletal or nonskeletal, or even more precisely, foraminiferal, corraline, etc.

- (ii) Mineral content, for example, aragonitic, calcitic, etc.
- (iii) Depositional environment, for example, pelagic, littoral, etc.

This terminology is of limited use to the geotechnical engineer since it gives little, if any, indication of the engineering behavior of a soil.

Fookes and Higginbottom (1975) were perhaps the first to attempt classifying carbonate soils from an engineering perspective. They used the following four criteria for classifying carbonate sediments:

- (i) Carbonate content
- (ii) Degree of induration
- (iii) Particle size
- (iv) Origin of carbonate material.

According to Datta et al. (1982), the classification system shown in Table 2.4 covers all types of carbonate sediments from carbonate mud to carbonate gravel, from unindurated carbonate soil to hard carbonate rock, and from impure carbonate sediments to pure carbonate sediments. Clark and Walker (1977) slightly modified this scheme of classification using the same four variables and presented it in a more systematic manner as shown in Table 2.5.

The systems of classification proposed by Fookes and Higginbottom (1975) and Clark and Walker (1977) are excellent schemes that encompass the entire spectrum of carbonate soils and rocks. According to Datta et al. (1982), these classification systems cover all possible calcareous soils, but they lack the judgment of the

Table 2.4: Proposed Classification of Pure Carbonate Sediments for Engineering Purposes (from Fookes and Higginbottom, 1975).

Particulate Carbonate Deposits (increasing grain size →)					Non-particulate or Massive Carbonate Deposits	
CARBONATE MUD 0.002 mm.	CARBONATE SILT 0.06 mm.	CARBONATE SAND		CARBONATE GRAVEL	No undurated representatives	
		Bioclastic Sand (organic)	Oolite Sand (inorganic)	Shell Gravel (organic)		Coral Gravel (organic)
CARBONATE MUDSTONE (calcisiltite)	CARBONATE SILTSTONE or CHALK (calcisiltite)	CARBONATE SANDSTONE or CALCARENITE		CARBONATE CONGLOMERATE (or Breccia if angular/calcirudite)		CORALLINE OR ALGAL REEF LIMESTONE
		Bioclastic C. Sct. or Calcarenite (microcoquina)	Oolite C. Sct. or Calcarenite	Shell Conglit. or Breccia (coquina)	Coral Conglit. or Breccia	
FINE - GRAINED LIMESTONE		DETITAL LIMESTONE		CONGLOMERATIC LIMESTONE (Breccoid limestone if angular)		
		Bioclastic Limestone	Oolite Limestone	Shelly	Coralline	
CRYSTALLINE LIMESTONE OR MARBLE (tends towards uniformity of grain size and loss of original texture)						

(established alternative names are in square brackets)

Table 2.5: Proposed Classification Chart for Description of Middle Eastern Sedimentary Rocks (from Clark and Walker, 1977)

ADDITIONAL DESCRIPTIVE TERMS BASED ON ORIGIN OF CONSTITUENT PARTICLES							TOTAL CARBONATE CONTENT % (Constituent particles plus matrix)
DEGREE OF INDURATION	APPROXIMATE UNCONFINED COMPRESSIVE STRENGTH	NOT DISCERNIBLE		BIOCLASTIC (Organic)	OOOLITE (Inorganic)	SHELL (Organic) ALGAL PISOLITES (Inorganic)	
		INCREASING GRAIN SIZE OF PARTICULATE DEPOSIT					
		0.002 mm	0.060 mm	2 mm	60 mm		
Non-Indurated	Very soft to hard (<36 to >300 kN/m^2)	CARBONATE MUD	CARBONATE SILT	CARBONATE SAND	CARBONATE GRAVEL	90%	
		Clayey CARBONATE MUD	Siliceous CARBONATE SILT (1)	Siliceous CARBONATE SAND (1)	Mixed carbonate and non-carbonate GRAVEL (2)	50%	
		Calcareous CLAY	Calcareous SILT (1)	Calcareous silica SAND (1)	GRAVEL	10%	
		CLAY	SILT	silica SAND		90%	
Slightly Indurated	Hard to Moderately Weak (0.3 to 12.5 MN/m^2)	CALCILUTITE (carb. clayst.)	CALCISILTITE (carb. siltst.)	CALCARENITE (carb. sandst.)	CALCIRUDITE (carb. conglom. or breccia)	90%	
		Clayey CALCILUTITE	Siliceous CALCISILTITE	Siliceous CALCARENITE	Conglomeratic CALCIRUDITE (2)	50%	
		Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE	10%	
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA	90%	
Moderately Indurated	Moderately strong to very strong (12.5 to 100 MN/m^2)	Fine-grained Argillaceous LIMESTONE	Fine-grained Siliceous LIMESTONE	Detrital LIMESTONE	Conglomeratic LIMESTONE (2)	50%	
		Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE	10%	
		CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE OR BRECCIA	50%	
						10%	
Highly Indurated	Strong to extremely Strong (70 to > 200 MN/m^2)	CRYSTALLINE LIMESTONE OR MARBLE (tends towards uniformity of grain size and loss of original texture)					50%
		Conventional metamorphic nomenclature applies in this section					

NOTES (1) Non-carbonate constituents are likely to be siliceous apart from local concentrations of minerals such as feldspar and mixed heavy minerals (Emery 1956).
 (2) In description the rough proportions of carbonate and non-carbonate constituents should be quoted and details of both the particle minerals and matrix minerals should be included.
 (3) The preferred lithological nomenclature has been shown in block capitals; alternatives have been given in brackets and these may be substituted in description if the need arises.
 (4) Calcareous is suggested as a general term to indicate the presence of unidentified carbonate. Where applicable, when mineral identification is possible calcareous referring to calcite or alternative objectives such as dolomitic, aragonitic, sideritic etc. should be used.

engineering behavior and use of these soils. They found that two carbonate soils, may have the same nomenclature using the above systems, and yet have entirely different particle crushing behavior and degree of cementation. Further, in these classification systems, the above mentioned authors have neither specifically stated the relevance of, nor they have attempted to incorporate the usual classification parameters like gradation and plasticity.

Datta et al (1982) stated the following deficiencies for the existing classification system:

1. There is not yet a method for readily identifying and quantifying the degree or uniformity of cementation.
2. There is not yet a simple method for quantitatively assessing the susceptibility to crushing of carbonate particles since visual or microscopic study of particle characteristics provides only qualitative ideas.
3. There is not yet a clear understanding of how different non-carbonate components of soils are influenced by the presence of carbonate material. Carbonate content was initially thought to be a useful index. However, its use is limited, since the amount of carbonate material present, is not the only factor controlling the engineering behavior, but the nature of non-carbonate material often plays a significant if not the dominant role.

In view of these limitations, Datta et al. (1982) gave suggestions for the pertinent information to be recorded for the carbonate soils as shown in Table 2.6. The information includes the cementation, grain-size distribution and plasticity, nature

Table 2.6: Proposed System of Description for Calcareous Soils (from Datta et al. 1982)

Description of ..	Remarks
1. Cementation (a) No cementation (b) Weak cementation (c) Strong cementation (i) uniform (ii) partial	the soil has a soft rock-like appearance. Unconfined compressive strength should be indicated the soil contains cemented aggregates - this should be noted
2. Grain Size Distribution (GSD) and Plasticity (a) Grain size distribution (b) Plasticity	for strongly cemented soils, GSD is not very relevant; for uniform cementation, size of constituent particles should be indicated; for partial cementation, GSD of soil after removing aggregates should be indicated and size and proportion of aggregates noted separately for fine-grained soils in which intraparticle voids cause error in GSD and Atterberg limits, field classification procedures may be used for providing the relevant information in a qualitative sense
3. Nature of Carbonate Component (a) Carbonate content (b) Particle size of carbonate material (c) Particle characteristics and origin (d) Mineralogy (e) Geologic name	soils having more than 30% carbonate content should be termed as carbonate soils the carbonate content in the sand and in the silt-clay fractions should be determined separately and indicated. Microscopic studies mentioned below will also give information about particle size microscopic studies - optical microscope for sands and scanning electron microscope for fine-grained soils - should be conducted. Presence of thin-walled material and intraparticle voids should be highlighted X-ray diffraction analysis should be performed if possible to identify, the geologic name may be indicated
4. Nature of Noncarbonate Component (a) Particle size (b) Particle characteristics (c) Mineralogy	information on noncarbonate material is determined by dissolving the carbonate material in HCl, separating the remaining soil, and conducting the following tests on it grain size distribution analysis microscopic studies X-ray diffraction analysis

of carbonate component and nature of non-carbonate component.

In Algeria, the classification of the calcrete was given by Horta (1979). Accordingly, the original Unified Soil Classification System (USCS) ranged calcretes with laterites as silty gravels and sands, GM and SM, but the representative points of the Atterberg limits for calcrete soils very often fall above the Casagrande line (A-line). Usual classification tests (gradation analysis and Atterberg limits) have limited significance when applied to calcretes. The determination of the carbonate content of the minus No. 40 sieve (0.4 mm) fraction is an important characteristic (Horta, 1979). Accordingly, he defined new symbols for calcretes. For example, SE for calcrete sands, which are defined as sands containing more than 40% carbonate in their fraction passing the 0.4 mm sieve, while calcrete gravels (GE), are defined as gravels containing more than 60% carbonate in their fraction passing the 0.4 mm sieve.

2.4 Stabilization of Marls in Eastern Saudi Arabia

In eastern Saudi Arabia, marl is one of the prominent soils being used extensively as foundation materials for pavements and structures. Marl, being a marginal and water sensitive soil, cannot be utilized as a construction material in its natural form, as many failure cases have been observed in highways. There have been some attempts in the past to stabilize these materials and to improve their engineering properties. Additives, such as sand, asphalt, and cement were investigated on very limited bases. The following sections present the major findings of these investigations.

2.4.1 Stabilization of Marl with Sand

Sand is usually added to other soils to improve their gradation and their resistance to moisture variations. Mineralogically, sand is mostly quartz, which is considered to be the most resistant soil to chemical and mechanical degradation. Thus, the water sensitivity of the marl could be reduced when it is mixed with sand. Qahwash (1989) prepared marl-sand mixes by blending marl from a specific site with three different types of sand at different weight percentages. The effect of sand content on the uniformity coefficient, specific gravity and plasticity index has been studied. His results indicated a decrease in the above parameters as the sand content increases.

The moisture-density relationship for the sand-marl mixes showed that the increase in sand content resulted in a decrease in the optimum moisture content (OMC) and an increase in the maximum dry density. The decrease in OMC is attributed to the replacement of fines in the mixtures with sand. The CBR test was used to evaluate the bearing characteristics of the sand-marl mixtures, and the results indicated that a mixture of 70% sand and 30% marl gave a sharp increase in the CBR values. Furthermore, the swelling characteristics were reduced significantly as more sand was added into the marl, until swelling became negligible at 70% sand content.

2.4.2 Stabilization of Marl with Bituminous Materials

Asphalt and emulsion are usually used to stabilize soils by providing cohesion among the particles and depriving them from their water affinity. Asphalt is a petroleum by-product consisting mainly of hydrocarbons. As a stabilizer, asphalt

provides an adhesive coating to particles and thus provides cohesion between the particles. It also deprives them from their water affinity and therefore promotes stability of the asphalt soil mixes. Asphalt has been used for the stabilization of marl as well as other soils. Emulsified asphalt (mixture of asphalt and water, in the presence of an emulsifying agent) can be used to treat most materials ranging from coarse aggregates to fine sands and clays without heating; and also without causing pollution. It has a better resistance to stripping than asphalt cement, especially when marginal aggregates, such as limestone, is used (Al-Abdul Wahhab and Arora, 1988).

Arora and Arabiat (1986) conducted a laboratory evaluation of the asphalt emulsion-treated mixtures for use in base courses for low-volume desert roads in Saudi Arabia. They used a mixture of marl and 30% wind-blown dune sand to meet the criterion recommended by Asphalt Institute (1980) for treating silty sands with asphalt emulsion for road bases. A locally produced, slow setting, cationic emulsion, CSS-1h, was used. Portland cement was also added in small quantities to enhance the development of early strength and resist water-induced damage. Emulsion-treated sand-marl mixtures were found appropriate to be used for low volume roads.

Al-Abdul Wahhab and Hicks (1988) used emulsified asphalt-treated marl mixes. The designed mixes were subjected to a series of laboratory testing to evaluate their engineering properties and to predict their behavior under field-simulated conditions. The optimized mixes were subjected to a series of dynamic tests, to simulate traffic loading. Laboratory tests were carried out to predict the two

major distresses associated with pavement failures, fatigue cracking and rutting. Systematic procedures were followed to utilize the laboratory results in the design of road sections suitable for local Saudi conditions. The results of the pavement structural analysis were presented in the form of design charts. In these charts, the total traffic in the form of equivalent 18 kip axle load is plotted versus stabilized layer thickness for different CBR values of the subgrade. It was noted that the fatigue criterion is the controlling factor in the case of marl emulsion, which also has the capability to withstand in-service temperature of more than 50°C.

Al-Abdul Wahhab and Arora (1988) used CSS-1h emulsified asphalt to stabilize marl. Portland cement (2% and 5%) was conjointly added for early strength enhancement. In the evaluation process, diametrical resilient modulus test and the indirect tensile test were used. They concluded that marl emulsion mixes tend to give higher stability due to the capability of withstanding in-service temperatures of more than 55°C with reasonably high modulus values.

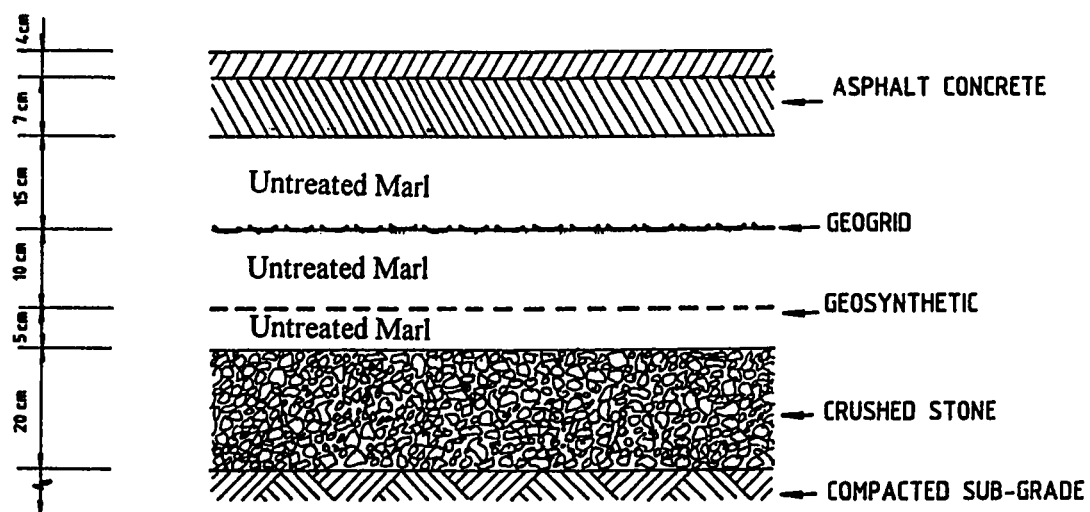
Al-Abdul Wahhab and Abduljawad (1989) stabilized a marl soil from eastern Saudi Arabia using two types of liquid asphalt; namely cationic slow setting emulsified asphalt (CSS-1h) and medium curing cutback asphalt (MC-70). Both lime and cement were added in small percentages to the mix in order to accelerate the early strength gain and to reduce the stability loss. Triaxial tests were performed on cylindrical samples to evaluate the effects of confining pressure, portland cement and lime additions, asphalt type and saturation. They concluded that increasing the cement or lime percentage will increase the strength, this increase is more significant in the case of portland cement. Soaking the untreated samples

resulted in a total collapse. However, the addition of lime to emulsion-treated marl increases the shear strength slightly, the strength gain was reduced when the lime was increased from 2% to 4%. Soaked shear strength for the treated soil was less than the dry strength due to the softening produced by water. Emulsified asphalt yielded higher shear strength values when 4% portland cement was used.

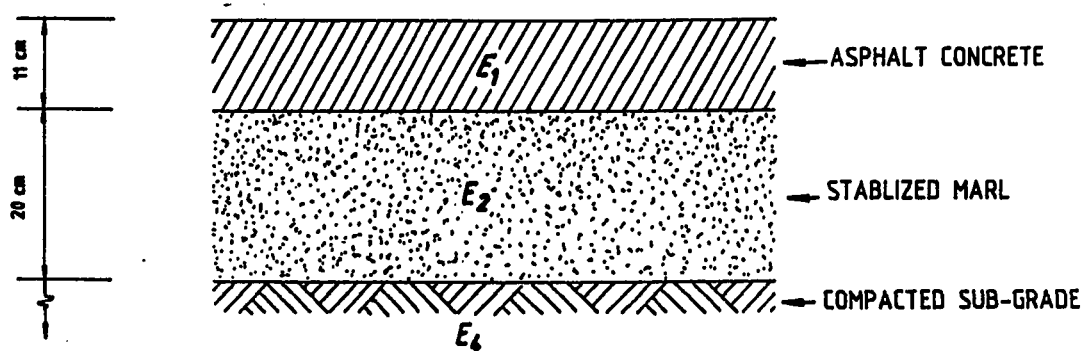
2.4.3 Stabilization of Marl with Cement

The efforts, so far, to stabilize marl using asphalt, sand and cement were only of academic value. No effort was directed to develop a comprehensive parametric investigation that can be used in the field. Although the stabilizers used improved the quality of the marl soil, the previous research lacked field implementation of these agents. To the author's knowledge, no case histories of field trials have been reported in the literature.

Recently, Aiban et al. (1995) have performed a field trial of marl stabilization using 4% cement on one of the main roads in Dammam industrial area, eastern Saudi Arabia. The stabilization program was carried out on a severely damaged paved road where marl was used as a base course. Although construction was properly carried out, the road was deteriorated three months after its construction. Typical cross-sections of the cement stabilized and the untreated original (unstabilized) parts of the road are shown in Fig. 2.6. The modulus of elasticity (E), was evaluated using back calculation technique from the *in-situ* Dynaflect testing. A summary of the results is shown in Table 2.7. These readings were taken 7 months after the commencement of stabilization. Considerable improvement in the E values of treated sections can be noted in comparison to the unstabilized



(a)



(b)

Figure 2.6: Typical Cross-section of the Road in Dammam Area a) Unstabilized Section b) Stabilized Section (After Aiban et al., 1995).

Table 2.7: Summary of the Back Calculation for Dynaflect Testing at SASCO Site, Dammam Industrial Area (Aiban et al., 1995)

Date	Section No.**	E ₂ for base course (MPa)	Remarks
12-4-94*	1-1	394.7	Untreated
	1-2	180.8	Untreated
	2-5	196.7	Untreated
06-06-94	1-1	140.1	Untreated
	2-1	1682.9	Treated with 4% Cement
	2-2	252.5	Untreated
	2-4	728.6	Treated with 4% Cement
26-10-94	1-1	431.3	Untreated
	2-1	2863.5	Treated with 4% Cement
	2-2	103.5	Untreated
	2-3	139.4	Untreated
	2-4	2851.1	Treated with 4% Cement

* Tested before treatment (date of treatment : May 2, 1994)

** Sections are shown in Fig. 2.7.

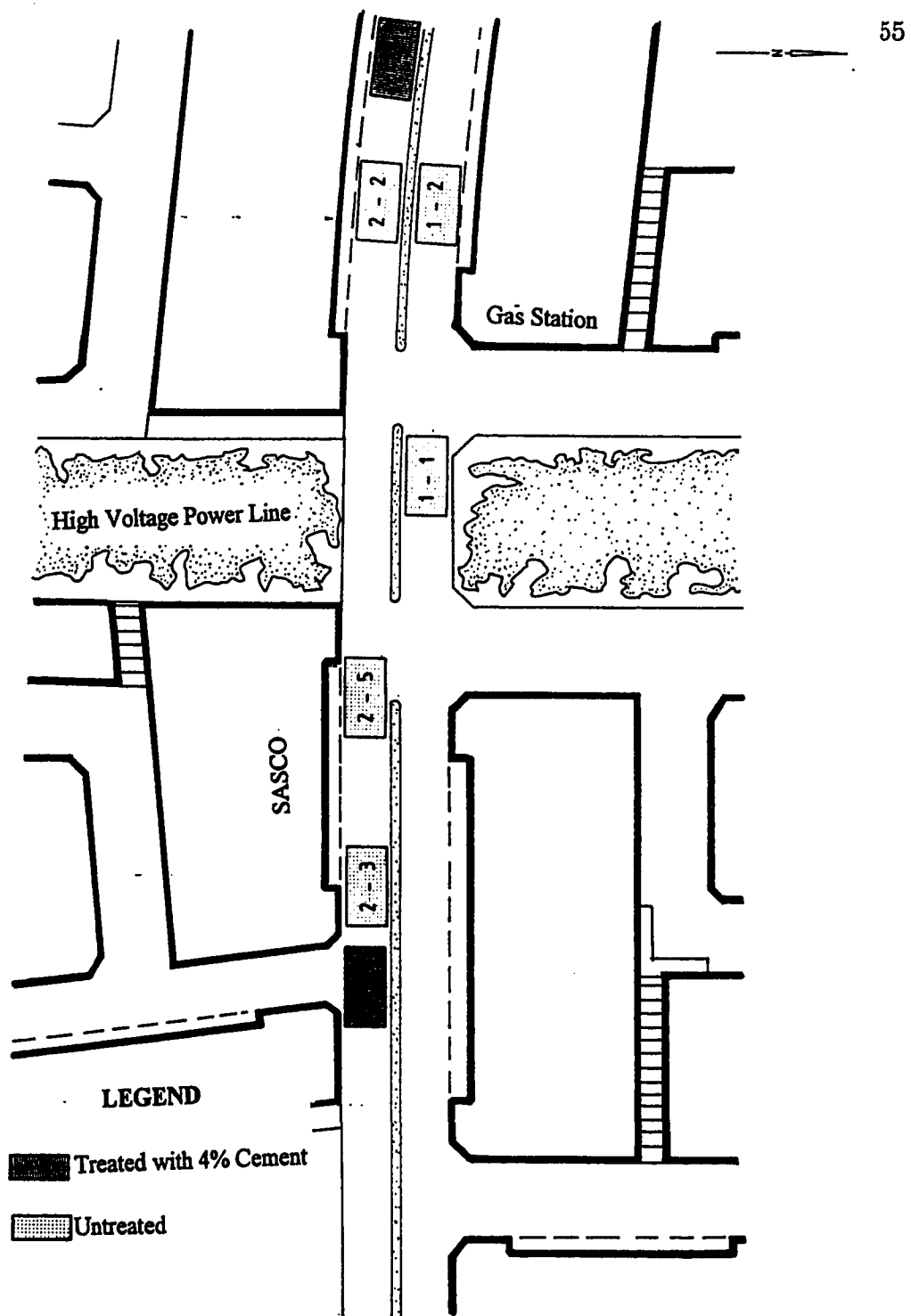


Figure 2.7: Site Plan Showing Stabilized and Unstabilized Sections of the Road in Dammam Industrial Area (Aiban et al., 1995).

sections. None of the stabilized sections have shown any signs of cracking or rutting after almost two year of service.

Comparison of the performance of the stabilized sections with that of the untreated ones where both are exposed to the same traffic volume indicates that the untreated ones are severely damaged and the asphalt layers are completely removed from the surfaces.

In light of the literature review, and the lack of a comprehensive study that accounts for most of the field conditions, it is strongly felt that there is a dire need of a comprehensive and practical research to characterize the marl soils from an engineering point of view. Based on the engineering characterization, and the deficiencies present in these soils, an improvement program was required to enhance the engineering behavior of these soils. The improvement techniques accounted for the deficiencies in both strength and durability and be practical and economically feasible. Moreover, these techniques take into account the local conditions in eastern Saudi Arabia.

Chapter 3

Experimental Work

The lack of enough information regarding the characteristics and behavior of calcareous soils combined with their poor performance necessitate further investigation on these soils. This experimental program was devised to study these soils and define the problems associated with them. The first phase of the experimental program comprised of collection of samples from different sites. In the second phase, these samples were characterized using ASTM and AASHTO standard tests. Based on the results obtained in the second phase, some of these soils were selected for further “detailed” characterization and stabilization and constitute the third phase. The experimental program was executed in a way to fulfill the objectives cited in Section 1.4. The flow chart for the experimental program is shown in Fig. 3.1.

3.1 Collection of Soil Samples

Sufficient and representative samples of the marl soils from different marl sources in the Eastern Province were collected. Some of these sources are still being in use for different construction purposes. However, some others were depleted and are not being utilized any more. A total of 24 marl samples were collected from dif-

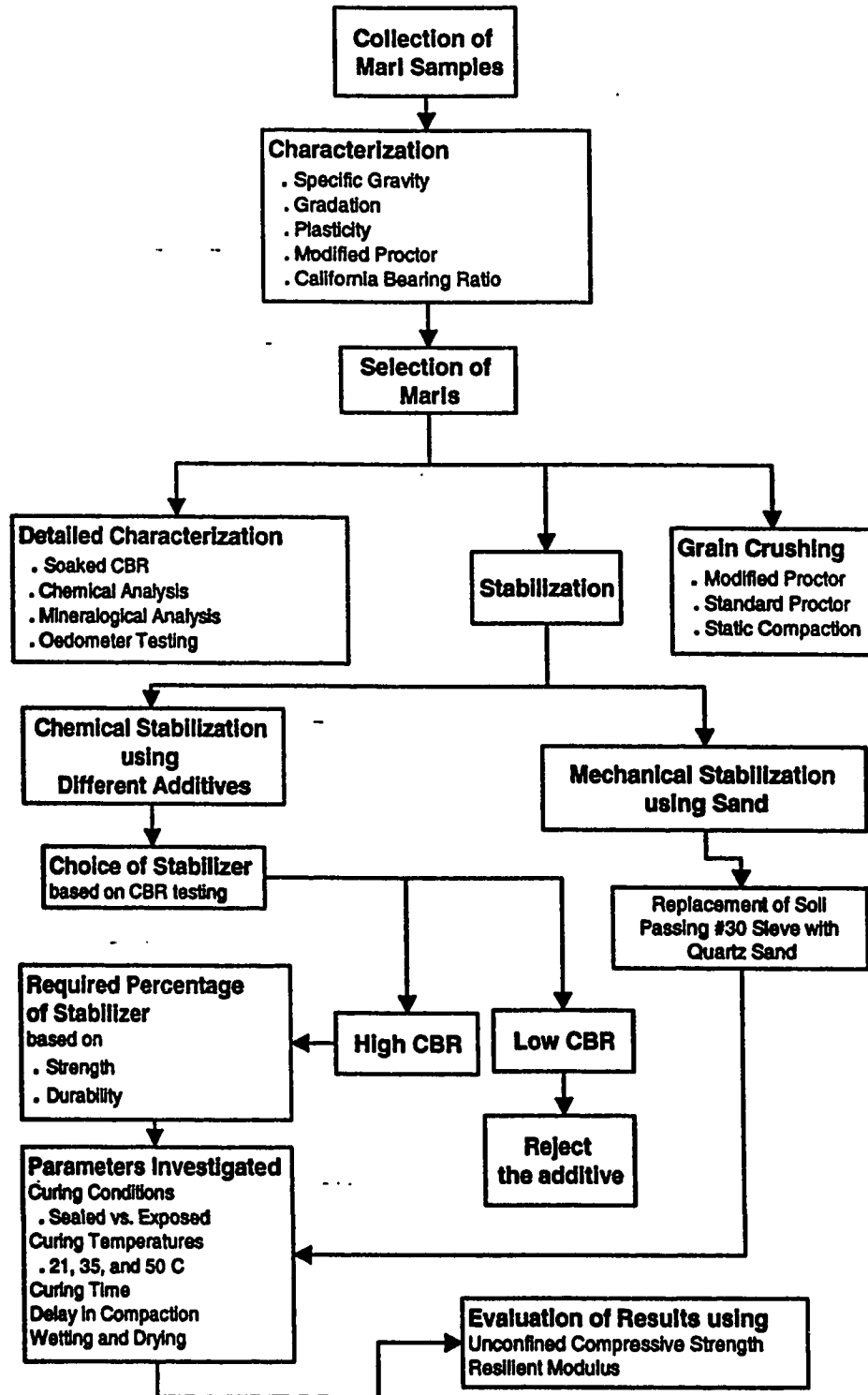


Figure 3.1: Flow Chart for the Experimental Program.

ferent borrow pits and from different construction projects. These were obtained either from existing piles of soil, or from crushed base course materials. The piles of soil were excavated by bulldozers to be used as a construction material. Some samples were also collected from the depleted marl soil sources. The intention of collecting samples from these areas was to make a comparison between the existing and the depleted sources.

The collected samples were labeled according to their site locations. These samples were then brought to the laboratory for testing. Before any testing, sieving of the gravel fraction was carried out and the material retained on 2 in. (50.8 mm) sieve was excluded. The material retained on each sieve was collected in bags and labeled accordingly. Upon testing, each soil was reconstituted according to its natural gradation or to a specific base course gradation. The quantity of the samples collected was enough to do the tests necessary for the preliminary characterization. These tests included: specific gravity, Atterberg limits, sieve analysis, modified Proctor, and California Bearing Ratio (CBR).

3.1.1 Location and Description of Marl Samples

The marl samples were collected from a total of 24 different locations at 7 different areas. The areas were Abu-Hadriyah, Abqaiq, Dhahran, Baggah, Hofuf, Shedgum, and Ain Dar; as shown in Fig. 3.2. Sometimes, the samples were collected from several borrow pits and locations within the same areas. This was due to the variability of materials both horizontally and vertically. Furthermore, different contractors were using soils obtained from different borrow pits within the same

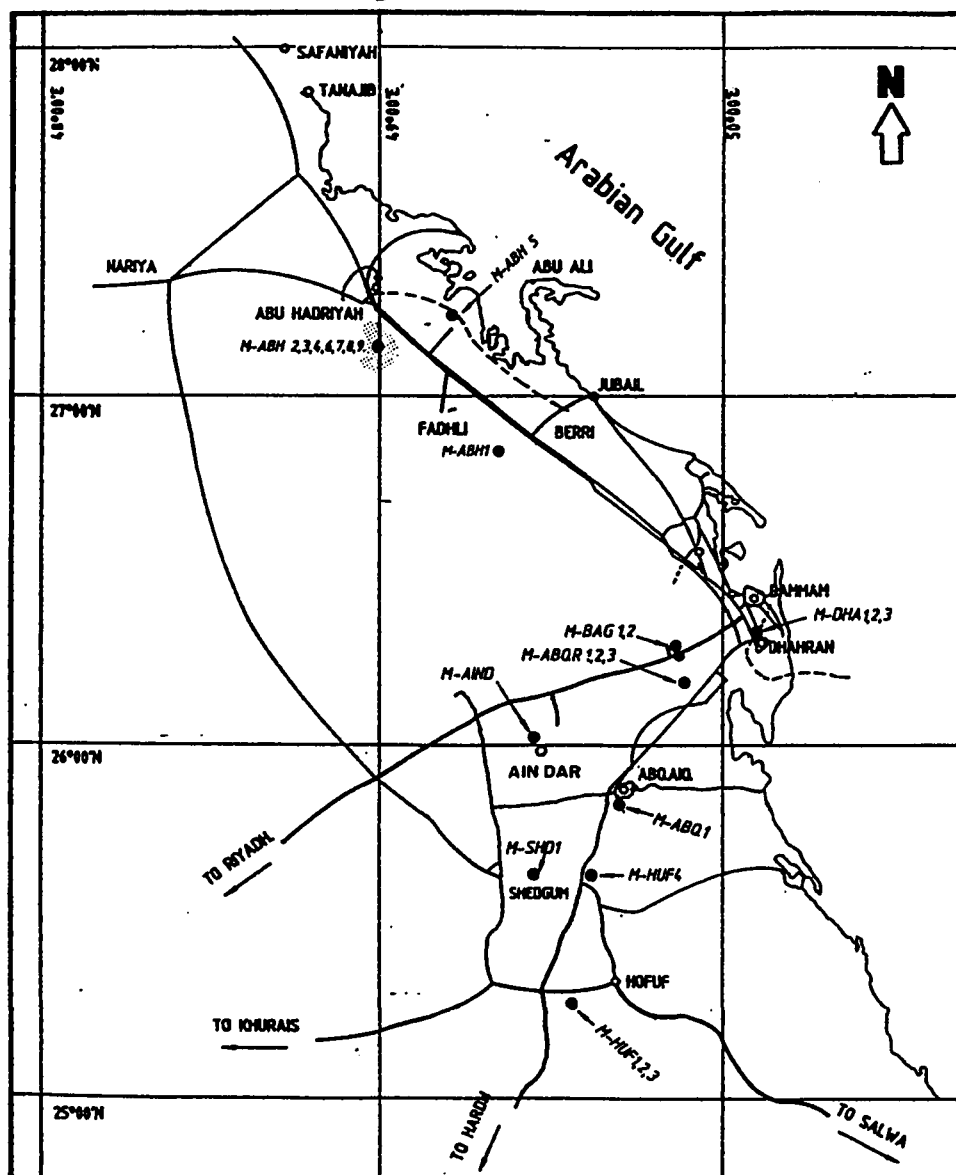


Figure 3.2: Vicinity Map for Marl Locations.

area. The samples were collected from different exposed layers of the borrow pits. Notes were taken down regarding the characteristics of each area and the material and the layering observed during the collection process. The impurities in the marl layers were observed and representative samples of some of these impurities were collected (Aiban et al., 1995a). The site location and soil description of various marls are given in the following paragraphs. Specific legends are given to each marl sample, and are going to be used hereafter.

Abu Hadriyah Marl (M-ABH1)

The borrow area is located to the west of Dammam-Abu Hadriyah Highway, about 116 km from Dhahran. It is near the ARAMCO Reservation Area for Tapline, very close to the Tapline Road. The site is a borrow area for Al-Ajeinah Company, and it is elevated compared to the surrounding area. A crystallized calcite layer, about 50 cm in thickness was located within a few meters (3 to 5 m) from the ground level. Light greenish clay was present in the borrow area. Different colors were observed in the calcite layer as well as in other layers. The fossiliferous nature of the borrow area reveals that the formation is of marine origin.

Abu Hadriyah Marl (M-ABH2)

The site is located about 10 km to the west of Dammam-Abu Hadriyah Highway, about 130 km from Dammam. It is the main borrow area for Abdullah A. Al-Dossary Company. The soil from this borrow area is accepted by the Dammam Municipality for its maintenance and construction projects. The borrow area is 10 to 12 m below the ground surface. The profile of the borrow area showed a large variability in the material with different layers. Well-defined and distinct

layers of sound limestone, pure and relatively loose sand, loose chalky material, soft limestone were clearly visible. The marl of this area was obtained from a 2 to 3 m layer which was located at a depth of 10 m below the ground surface. The soil sampled was light yellowish in color. Sandstone and some softer material were found to be fossiliferous in nature.

Abu Hadriyah Marl (M-ABH3)

The site is near the Al-Dossary borrow area (M-ABH2), and is a borrow area for Al-Derbas Company. The samples were collected from a depth of 2 m from the surface. The marl collected from the area was a mixture of light yellowish calcareous sediment and light gray stones. Eolian sand deposits were observed near the borrow area. The marl retrieved by this company is ensured to be free from sand. The soil from this borrow pit is used as a base course in the construction of some of the new streets in the Dammam area.

Abu Hadriyah Marl (M-ABH4)

The sample was obtained from the same area as that for M-ABH3. The coarse aggregate was very sound, dark gray sandstone. Normally, the higher quality aggregates produced in the quarries are used in concrete construction. However, they are also reserved for some special purposes such as the base course construction requirement by the Ministry of Communication (MOC). MOC requires high quality aggregates for their pavement construction.

Abu Hadriyah Marl (M-ABH5)

The borrow area is located opposite to the M-ABH2 and M-ABH3 sites. It is to the east of the Dammam-Abu Hadriyah Highway, towards the coast. The profile consisted of 3 to 5 m thick layers of sand underlain by 0.5 to 1.0 m thick layers of greenish, highly plastic clay. The clay was interbedded with relatively thick (10-15 cm) layers of gypsum with very thin seams of clay. The marl sample was obtained from a depth of 5 to 6 m from the ground surface. The color of this marl is light gray and yellowish. There were some soft materials of dark yellow/orange color, developed inside the voids present in the stones.

Abu Hadriyah Marl (M-ABH6 and M-ABH7)

The borrow area is located adjacent to that of Al-Dossary borrow area (M-ABH2). It was a newly utilized borrow area for the use of Al-Osais Company. The profile at the surface consisted of two distinct layers. The bottom one (M-ABH6) consists of milky colored sandy material, while the top layer (M-ABH7) is brown to golden in color. The brownish material is clayey in nature.

Abu Hadriyah Marl (M-ABH8)

The marl is obtained from a construction site. It is light yellow in color. This marl is being used as a base course construction in the Dammam Industrial Area. It was used for construction of some of the main streets in the Dammam area constructed by Al-Osais company.

Abu Hadriyah Marl (M-ABH9)

This material is being used by the Dammam Municipality for the construction and maintenance projects. The material was taken from the area of M-ABH2, but being blended as a base course. The marl was reconstituted to a specific base course gradation in the KFUPM Geotechnical Laboratory, to compare its behavior with the original marl (M-ABH2). The gradation to which the soil was reconstituted is shown in Fig. 3.3.

Abqaiq Marl (M-ABQ1)

The marl was obtained from Abqaiq area. It is white in color with little brown and orange colored inclusions. This marl is extensively used by Saudi-ARAMCO for the base course construction. It is considered as one of the best materials in the area. Aggregated dolomite is the main constituent of this marl, while chert and gypsum are present as inclusions.

Dhahran-Abqaiq Road Marl (M-ABQR1)

The borrow area for this marl is located about 5 km to the north of Dhahran-Abqaiq Highway and 46 km from Dhahran (KFUPM). The marl obtained from this area is whitish in color. The trace minerals are flint and chert of milky and brown colors. The flint and chert are of boulder size. Light brown/orange traces of aggregated materials were also present. Crystallized calcite and gypsum were observed in the borrow area. The borrow area was surrounded by few medium-sized sand dunes.

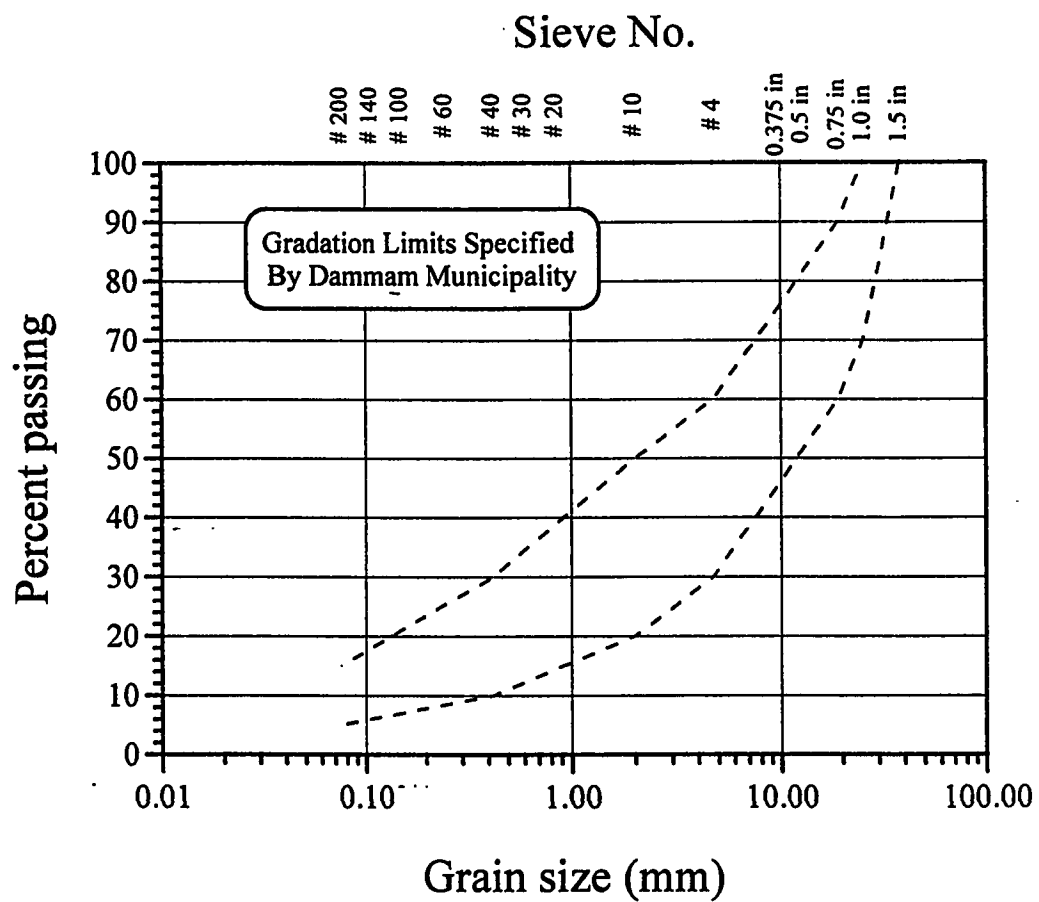


Figure 3.3: Base Course Gradation Specified by Dammam Municipality.

Dhahran-Abqaiq Road Marl (M-ABQR2)

The borrow area of this marl was located 2 to 3 km to the northeast of M-ABQR1. It is adjacent to the Dammam Municipality dump site. The marl, in general, was the same as that obtained from M-ABQR1. Dark colored chert and flint are present in larger quantities.

Dhahran-Abqaiq Road Marl (M-ABQR3)

This marl was obtained from Al-Noa'imi crusher, which is located about 8 km to the North of Dhahran-Abqaiq Highway, about 40 km from Dhahran (KFUPM). The marl was being crushed to a specified base course gradation. It is light, milky in color, with some gray aggregates. This marl was used as the base course material for the construction and maintenance of the airport and other projects.

Ain Dar Marl (M-AIND1)

The marl was obtained from Ain Dar area. It was brought to the laboratory by a contractor. This marl is white/light yellow in color.

Baggah West Marl (M-BAG1)

The area of collection is located to the west (1 km) of Dammam-Riyadh Expressway, near Station 1300+00. The marl was obtained as a bulk sample from a surface borrow pit, which was excavated by a bulldozer. The marl is white with light/dark brown hard and heavy inclusions. The marl is fossiliferous in nature. The site has some medium sand dunes with little vegetation.

Baggah East Marl (M-BAG2)

The marl was obtained from a site located to the east (1 km) of Dammam-Riyadh Expressway, near Station 1300+00, facing M-BAG1. The soil was collected from an existing pile in the area. The area had some medium sand dunes with no vegetation. It had very hard internal casts of different sizes ranging from 2 to about 10 cm.

The marl (M-BAG1 and M-BAG2) has been used by Al-Ajeinah Company for the maintenance of the Dammam-Riyadh Expressway. The soil contains traces of light pink material which could be ferrous in nature.

Dhahran "Rabwah" Marl (M-DHA1 and M-DHA2)

The borrow area (Rabwah) is located west of the Dhahran-Dammam Highway. The soil was obtained from the northern part of the Rabwah residential area. A vertical cut was made in a hillock, exposing about 12 m of the subsurface profile. The area was planned to be used for residential purposes.

The exposed profile consisted of sound limestone, large boulders of aragonite (1 to 2 m), a thick layer of cracked, highly plastic, brown material. Different colors, including white, milky, brown, yellow, orange, and pink, were observed. Large vertical voids in the layers were filled with sand and other aggregates. During excavation, highly plastic material was observed to be mixed with the marl. In the laboratory, M-DHA2 was reconstituted as M-DHA1 and 20% of the plastic material.

Dhahran Marl (M-DHA3)

The site is located to the east of Rabwah (east of M-DHA1) and to the east of Dhahran-Dammam Highway, 1 km away from M-DHA1. The area is to be reclaimed for residential purposes. The soil is almost the same as M-DHA1 except for the absence of aragonite.

Hofuf Marl "Base Course" (M-HOF1)

This marl was obtained from a contractor (Gulf Company). The material was taken from a yard about 10 km from Hofuf on Hofuf-Riyadh Highway. There were two different piles consisting of fine and coarse materials. These are usually reconstituted on weight basis in a ratio of 35% and 65%, respectively. The fine material was light yellow/brown in color while that of the coarse material is brown and light gray. Crystallized calcite and gypsum were almost absent in the material.

Hofuf Marl "Base Course" (M-HOF2)

The soil was taken from the same yard as that of M-HOF1. The pile from which the soil sample was taken was already blended by the company. Therefore, the marl constituents were the same as M-HOF1 but the gradation was different. The soil was reconstituted to the gradation of the marl obtained from the pile.

Hofuf Marl (M-HOF3)

This marl was obtained from a borrow area, about 500 m to the west of Hofuf-Riyadh Highway, about 10 km from Hofuf. The bulk soil sample was obtained from an existing pile. The pile was excavated using bulldozers and the material is brown in color and plastic in nature.

Hofuf Marl (M-HOF4)

The marl was obtained from an area to the north of the Hofuf cement factory. The area of sampling was 200 m to the south of Dammam-Hofuf Highway. The material was collected from an existing pile. The whitish marl has excessive fines and gypsum nodules and mushes.

Shedgum Marl (M-SHD1)

This marl was obtained from a surface borrow pit, about 50 cm deep, near the Shedgum facility road. The marl is being used by Saudi-ARAMCO for construction purposes. The borrow area contained large boulders. No crystallized gypsum and calcite were observed. The marl has a pinkish color. This marl is part of the duricrust found in the Shedgum Plateau. The aggregates and fines obtained seem to be the weathered product of the duricrust. Before weathering, the aggregates might have been cemented by carbonates.

3.2 Characterization of Eastern Saudi Marls

The engineering behavior of a soil is determined by various standard index and strength parameters. The response of the soil to different tests is intended to predict its behavior in the field to verify whether it is suitable as a construction material by itself, or certain improvements are required to make it desirable for the construction. For this purpose, certain soil tests were performed on the marl samples collected from the different sites in the Eastern Province. These include the tests necessary for classification using plasticity and grain-size analysis. In addition, compaction and strength tests were performed on all samples using the modified Proctor compaction and California bearing ratio tests. These are the

basic requirements for the usage of a soil in highway construction. Specific gravity test was also included as a part of the preliminary tests.

3.2.1 Specific Gravity

The marly soils contain limestone as the main ingredient. Although the specific gravity is not a direct measure of the strength and other engineering properties of the soils, it is used as a parameter in the determination of some important properties of the soil such as void ratio, unit weight, soil particle-size determination using the hydrometer method. It is also used in the determination of the saturation of the soil during the consolidation process.

For each marl type, the soil passing ASTM Sieve No. 4 was used for the determination of specific gravity as per ASTM D 854. The average of triplicate specimens is taken as the specific gravity value of the soil.

3.2.2 Plasticity tests

The plasticity tests are usually performed on the soil passing ASTM No. 40 sieve. In this investigation, two sets of plasticity tests were performed; one on the material passing ASTM Sieve No. 40 and the other on No. 100. Liquid limit and plastic limit tests were performed according to ASTM D 423 and D 424, respectively. For some soils, it was not possible to get the required number of blows for the liquid limit test, even in the driest possible state, therefore, the liquid limit was reported as nil.

3.2.3 Grain-Size Distribution

Grain-size analysis was performed in three stages. First, the gravel-size fraction was sieved through a set of coarse sieves. The sieve sizes of 1 1/2" (38.1 mm), 1" (25.4 mm), 3/4" (19.0 mm), 1/2" (12.7 mm), 3/8" (9.53 mm) and ASTM No. 4 were used. The material retained on each sieve was collected and stored in bags.

The soil passing ASTM No. 4 sieve was subjected to both wet and dry sieving (ASTM D 422). These methods are also known as washed and unwashed sieving, respectively. A set of sieves including ASTM No. 10, 20, 30, 40, 60, 80, 100, 140 and 200 sieves was used for both wet and dry sieving. After dry sieving, the soil retained on each sieve was dried, if necessary, and weighed. These soil portions were collected in a dish and remixed. The same soil was also sieved in the wet process through the same set of sieves. Distilled water was used in the process. Soil portions retained on each sieve as well as passing the No. 200 sieve were dried in the oven and then weighed.

The soil portion passing sieve No. 200 in washed sieving was subjected to hydrometer analysis (ASTM D 422). In addition, calcium chloride solution of 0.001 M was used in the hydrometer testing. The calcium chloride solution does not dissolve carbonate particles, which is considered to be one of the factors causing misleading results in hydrometer testing in case of calcareous soils.

3.2.4 Moisture-Density Relationship

Proctor compaction test is used to provide a relationship between the molding moisture content and the dry density of a soil. Two types of Proctor tests are

used in practice; Standard Proctor test (ASTM D 698) and Modified Proctor compaction test (ASTM D 1557). The use of modern heavy compacting equipment in the field necessitates the use of the modified Proctor compaction test. Therefore, Modified Proctor test was adopted in the characterization of all marl soils.

In the modified Proctor compaction test, a hammer weighing 18 lb. (8.18 kg) is used to compact the soil in five layers in the CBR mold. CBR sample has a diameter of 6 in. (152 mm) and a height of 5 in. (127 mm). The height of fall of hammer is 18 in. (457 mm) and the number of blows per layer is 56. The soil retained on 3/4 in. (19.0 mm) and coarser sieves was replaced by an equivalent amount of soil passing 3/4 in. (19.0 mm) sieve and retained on #4 sieve. Before compaction, the soil was reconstituted either according to its natural gradation or to a specific base course gradation.

3.2.5 Unsoaked CBR and Clegg Hammer Tests

California Bearing Ratio (CBR) test is an adhoc and an arbitrary test. It has been commonly used in the structural design and evaluation of pavements. There is no direct relation between CBR value and the shear strength parameters, such as cohesion and angle of internal friction. It is basically a static test, while the loads generated by the traffic wheels are dynamic in nature. The results can only be used for the method of design for which the test was devised. In spite of these limitations, the test is recognized worldwide because of its simplicity and applicability. Therefore, it can easily be used to judge the material for use in pavement construction.

The samples prepared for the moisture-density relationship were subjected to CBR testing procedure (ASTM D 1883). Samples were tested immediately after preparation so that no moisture loss is permitted. For the preliminary characterization stage, only unsoaked CBR tests were performed. However, to simulate the field conditions in which the soil is flooded with water, soaked CBR test was adopted for the marls selected for detailed characterization. Flooding can either be from the ground water or from the rain water infiltrating the layers. After compacting the samples in the CBR mold, the samples were soaked in water by placing them in a water tank for 96 hours. At the end of the soaking period, the specimens were tested in the same way as the unsoaked ones.

Clegg Impact Hammer, shown in Fig. 3.4, is considered to be a practical alternative to the CBR test as it can be performed both in the field and in the laboratory (Khan et al., 1995). Moreover, it can be used for quality control in the field. In addition, it is cost effective and easy to operate. The apparatus consists of a compaction hammer with shape and size conforming to modified Proctor test, equipped with piezoelectric accelerometer, and connected to a digital measuring device (Asi et al., 1992). It measures the Clegg Impact Value (CIV) by the dynamic rebound of the soil against a standard weight falling from a standard height and it is a measure of the strength of the material (Khan et al., 1995). To get a correlation between CBR and CIV, the samples prepared for CBR tests were also tested using the Clegg Hammer. At the end of CBR test, the Clegg Hammer was used on the bottom face of the sample to get the corresponding CIV. Clegg Hammer used in this investigation, was manufactured by 'Controls Milano Italy' (model T 168-005).



Figure 3.4: Clegg Impact Hammer.

3.3 Detailed Characterization

Based on the results of the initial characterization, two marls were selected for stabilization. The marls selected for stabilization were subjected to additional tests in the detailed characterization to completely define the properties and behavior of marl under investigation. These tests were X-Ray Diffraction (XRD), chemical analysis, oedometer testing and soaked CBR.

3.3.1 X-Ray Diffraction (XRD) Analysis

X-Ray diffraction (XRD) analysis is a technique used to define the mineralogical composition of a material. Knowledge of the mineralogical and chemical composition of a soil is important and can help in understanding the behavior of the soil. The method is qualitative as it provides information about the type of existing minerals but it is semi-quantitative as it gives the percentages of crystalline minerals only.

For XRD analysis, the soil sample is broken into small particles by a ceramic mortar pestle and into powder form by agate mortar pestle. Sample is pressed into a sample holder. This method is called Powder Method of X-Ray diffraction. The crystals are present in a random orientation in the sample. A monochromatic (single-wavelength) beam of X-rays is made to strike a crystal. Since the crystal is composed of parallel planes, represented by Miller indices (hkl), X-rays striking these planes reflect at a certain incident angle θ . A series of reflections occurs

depending on the type of minerals present in the soil. This reflection takes place only if the incident beam makes an angle which satisfies Bragg's law. The reflected beam is recorded on a strip chart according to the intensity of the reflected beam.

Comparison of the intensity and the angle of incident (θ) with those of standard minerals gives the type and percentage of minerals present in the sample. The comparison of diffraction pattern is made with standard diffraction patterns for different phases established by the Joint Committee of Powder Diffraction Standards (JCPDS).

3.3.2 Chemical Analysis

Knowledge of the chemical composition of a material helps in predicting its behavior under different environmental conditions. Calcium carbonate content of the soil passing ASTM No. 40 sieve, (known as the calcium carbonate equivalent) is a useful index property of the calcareous soils (Horta, 1988). In some of the African countries, there are certain minimum limits for this index, and it is considered to be an alternative to the Plasticity Index (PI) (Horta, 1988). For marly soils, this index can be adopted, but there is a need for further investigation.

Chemical analysis of a soil includes elemental analysis and its reaction to various environmental conditions. The minerals constituting the soil are either chemically inert or active and may interact chemically with the pore fluids. Knowledge of the chemical interaction of the constituents of a soil is also important in the soil stabilization procedures.

The marls selected for detailed characterization were analyzed chemically, mainly for the presence of calcium and magnesium. The amount of calcium and magnesium is indicative of the presence of certain compounds such as carbonates, sulphates, chlorides, etc. The carbonates of calcium and magnesium are nothing but calcite and dolomite, respectively.

The Ethylenediaminetetracetic acid (EDTA) method (Bisque, 1961) is based on the principle that the amount of EDTA necessary to react with a given metal ion may be directly measured by adding the EDTA solution to a solution of the metal under conditions such that the end point of the reaction may be detected. Calcium and magnesium react quantitatively with EDTA. These elements can be differentiated as they react at different pH values.

The soil portions passing ASTM Sieves No. 40 and No. 100 were used for elemental analysis using the methods provided by Bisque (1961). For each sample, few grams of soil were placed in an oven at 110°C for one hour. The sample was allowed to cool in a desiccator at room temperature. 1 gm of oven-dried soil was placed in a 400 ml beaker. 150 ml of 3 N HCl was added to the beaker and the solution was held at 50 to 60°C for half an hour. The solution was then filtered through a #4 filter paper and collected in a 1000 ml volumetric flask. The filtrate so obtained was used for the EDTA analysis.

To determine the sum of calcium and magnesium, 100 ml of the filtrate was placed in a 400 ml beaker. To the beaker, 15 ml of the ammonium-ammonium-chloride buffer solution, 5 ml of the 10% hydroxyamine hydrochloride solution,

several crystals of sodium tartrate, and a small pinch of the indicator EBT (Eriochrome Black T) were added. Then, it was titrated against 0.025 M solution of EDTA. The end point was a change in color from dark purplish red to blue which did not fade. The quantity of solution utilized in ml multiplied by 0.025 provided the millimoles (mM) of calcium plus magnesium.

In order to determine the amount of calcium, 100 ml of the filtrate was placed in a 300 ml beaker. To this filtrate, 8 to 10 ml of the potassium hydroxide solution, 5 ml of the hydroxyl-amine hydrochloride solution, several crystals of sodium tartrate, and a pinch (app. 0.1 gm) of indicator (murexide) were added. The solution was mixed using a magnetic stirrer to produce a pink solution. It was then titrated using 0.025 M solution of EDTA till a blue color was obtained. The number of ml required for this titration multiplied by 0.025 gave the number millimoles of calcium in the filtrate.

3.3.3 Oedometer Testing

Oedometer testing quantifies the compressibility of a soil. The compressibility of soil results from the rearrangement, crushing, and elastic compression of soil grains, in addition to consolidation. Consolidation is defined as the gradual settlement of soil resulting from the expulsion of water from the pores; and it occurs in fine-grained, saturated or nearly saturated soils. In contrast, swelling is the increase in volume of soil due to adsorption of water by the soil particles. The swelling characteristics of soils is determined using oedometer testing.

The material passing ASTM sieve No. 4 from the selected marls was subjected

to percentage of swell and consolidation testing, according to ASTM D 4546 and ASTM D 2435, respectively. Soils were tested at the optimum moisture content, 95% of maximum dry density on the dry as well as on the wet sides of optimum moisture content. Each sample was compacted in the oedometer ring (70 mm in diameter by 19 mm in thickness) at the required moisture content and dry density. A surcharge load causing a stress of 6.9 kPa (1 psi) was applied on the sample. The sample was thereafter inundated with water and then allowed to swell under the surcharge. When the rate of change of the specimen height was negligible, the specimen was loaded and unloaded with a load increment ratio of 1 from a pressure of 25.5 kPa to a pressure of 1630 kPa. Each load increment was kept until the deformation rate became negligible. The height of sample at the end of each load increment was recorded using a data acquisition system (model No. TDS-301 Tokyo Sokki).

3.4 Stabilization of Marls

Stabilization of marl soils constitutes one of the primary objectives of this investigation. The stabilization technique(s) was decided on the basis of the marl behavior observed in the characterization stage. The marls selected for detailed stabilization were then optimized for the selected stabilization technique(s) taking into account certain parameters that are expected to affect the behavior of the stabilized soils.

3.4.1 Stabilization Techniques

Depending on the behavior of the marl soils, the following stabilization techniques were selected and applied:

Mechanical Stabilization Using Sand

Mechanical stabilization using sand, also known as "granular" stabilization, involves preparation of a mixture of soil-aggregate consisting of stone, gravel, sand, and silt-clay; so as to obtain a well-proportioned mixture of particles with continuous gradation (well-graded) and desired plasticity (Winterkorn and Pamukcu, 1991). This mixture is to be compacted to the maximum density to obtain high strength, stability, and durability under all conditions. For calcareous soils, the sand and silt-sized soil particles are mostly carbonate particles which are the skeletal remains of micro-organisms, as shown in Table 2.1. Consequently, these soils are susceptible to crushing upon compaction, dissolution in acidic waters, and offer less frictional resistance compared to those of quartz sand. Due to this poor performance of sand and silt-sized particles, a replacement of these soil particles with equivalent amount of quartz sand has been made.

In this technique, the soil passing ASTM Sieve #30 was replaced by an equivalent quantity of quartz sand, and termed 'modified marl'. CBR testing was used to evaluate the behavior of the original and modified marls. Further, the modified marl was also subjected to chemical stabilization.

Chemical Stabilization

Chemical stabilization consists of mixing the soil with one or a combination of chemical admixtures, for the general objectives of improving or controlling its volume stability, strength and stress-strain behavior, permeability, or durability (Winterkorn and Pamukcu, 1992). Admixtures can be in the form of powder, slurry, or liquid. The fundamental processes that take place in a chemically-

stabilized soil system are cementation and ion-exchange reactions, alteration of soil surface properties, plugging of voids, and coating the soil particles thereby binding them together (Winterkorn and Pamukcu, 1992). The most commonly used chemical admixtures are portland cement, lime, fly ash, and bitumen. The selection of an admixture depends on the physical and chemical conditions of the natural soil, workability of the mixture, economic and safety constraints, the function of the treated soil, and the conditions of construction. The water affinity and water-retention capacity, clay content, grain-size distribution, and the porosity are the soil parameters that affect the interaction of the admixture with the soil particles. Cement and lime have been used successfully for the stabilization of different soils with varying degrees of improvement. These two will be discussed in the following paragraphs.

Cement has been used to improve the strength of soils for almost the last 50 years. It can practically improve the strength and durability of any type of soil provided that the plasticity of the soil is not very high. Cement reacts with water to form cementitious materials that bind the soil particles together at their points of contacts. However, the presence of organic matter and sulphates in a soil hinder the hydration of cement.

Lime exists either as quick lime (CaO) or hydrated lime [$\text{Ca}(\text{OH})_2$]. When lime is added to a clayey soil, a number of reactions takes place. Some of these occur immediately such as base exchange while others occur slowly. Clay particles are negatively charged and surrounded by a number of cations which are usually exchangeable. Strongly-charged calcium ions replace the weaker ions of potassium,

magnesium, sodium, and hydrogen thereby reducing the plasticity of soil. The soil particles tend to agglomerate into large-sized particles, a process known as flocculation. Thus, soil-lime mixture becomes friable and workable. After the early reaction, the excess lime reacts with the aluminous and siliceous materials in the clayey soil in the presence of water. This results in the formation of cementitious gels, imparting strength and durability to the mixture. These pozzolanic reactions are long-term reactions and may take years to complete. Another possibility is the formation of calcium carbonate due to the absorption of carbon dioxide from the atmosphere. This clearly indicates that clay soils are most amenable to lime treatment. The presence of a certain minimum percentage of clay or other pozzolanic materials is essential for the lime to form cementitious gels.

Portland cement and hydrated lime were used as chemical additives for the selected marls. The appropriate chemical additive was thereafter selected based on the CBR test results. Moisture-CBR relationship was developed and compared for 5% additive content. Cement was found to be a better candidate than hydrated lime, both from strength as well as economy point of view.

3.4.2 Optimization of Marl Stabilization

Any type of improvement, to which a soil is subjected, causes an enhancement of certain properties. The level of improvement required varies from project to project and from soil to soil. The improvement does not only depend on the type and amount of stabilizer but also on the environmental conditions associated with a particular site and the construction procedures. Considering all conditions which contribute positively or negatively, an optimum level of stabilization should

be determined which, besides being economical, should also satisfy minimum requirements intended.

Marl is used as a construction material for base and sub-base of pavements. For pavement structures; strength, settlement and durability are the primary concerns. Strength of a stabilized material can be expressed in terms of unconfined compressive strength, CBR or resilient modulus and can be enhanced with certain chemical additives. There are, however, certain ranges of moisture and temperature for which the benefits are maximized.

The following parameters were selected for the optimization process:

1. Cement content
2. Wetting and drying
3. Curing conditions
4. Delay in compaction

The effects of these parameters on strength and durability is evaluated using:

1. Unconfined compressive strength
2. Resilient modulus
3. Durability

These parameters along with their importance in the optimization process are discussed in the following paragraphs.

Cement Content

In this investigation, cement content is defined as the percentage of the weight of cement to that of the dry soil plus cement. Because cement is an expensive material, there is a need to determine an optimum value; which depends on the soil type. Granular soils are the most suitable, as they do not require any pulverization. They mix well with cement and need less quantity of cement due to their relatively low specific surface area. On the other hand, fine-grained soils require high percentages of cement due to their large specific surface area. Furthermore, higher percentage of cement is required for highly plastic soils. Normally, the cement content ranges from 4 to 16 % by dry weight of soil (ACI Committee, 1990).

In this research, three different percentages of cement were used to optimize cement content for cement-modified marl. These were 3, 5, and 7 % in addition to the untreated soil (i.e. 0% cement). These percentages were used to get the moisture-density relationships of soil-cement mixes. The desired cement content was selected based on the unconfined compressive strength of cement-stabilized specimens.

Wetting and Drying

The soil-cement in the field is subjected to repeated wetting and drying due to moisture variation. Rise and fall of water table and seasonal variation of rainfall are responsible for these wetting and drying cycles. These cycles cause consequent shrinkage and expansion stresses in the cement-stabilized soils. To simulate the field conditions, cement stabilized marl samples were subjected to several wetting and drying cycles. Each cycle consisted of wetting the samples by soaking them

in a water tank and then drying them in the oven for a specific period. These procedures were performed according to ASTM D 559. In addition, a new procedure for this purpose is validated and will be proposed for further studies.

Curing Conditions

Portland cement reacts with water to produce cementitious material, known as cementing gel. This reaction is called cement hydration. Cement needs certain amount of water for its complete hydration. The hydration reactions are initially fast, but its rate decreases with time. In general, the rate is very high during the first 7 days, and depends on the ambient temperature and curing conditions.

In the field, the moisture loss is usually prevented by different methods. The exposed surface of the stabilized soil is either covered with a wet layer of sand, burlap, etc., or the surface is sealed by spraying curing compounds such as emulsified asphalt. Water is regularly sprinkled to supplement the moisture loss. To simulate field conditions, samples for unconfined compressive testing were subjected to various curing regimes. The moisture conditions were controlled either by sealing the sample in plastic wraps, or by keeping the specimens uncovered (exposed conditions). The sealed-samples were wrapped with 5 layers of plastic sheet, to ensure that no moisture loss takes place. The samples for the exposed conditions were not sealed at all.

Due to the variations of the air temperature on daily and seasonal basis, it was hard to simulate the field conditions accurately. According to Al-Abdul Wahhab et al. (1994), the average seven day maximum temperature in Dhahran and Al-

Hasa area of the Eastern Province has been recorded to be 46.3°C and 47.6°C , respectively. Therefore, to get an idea about the strength gain of the soil-cement specimens with the variation in temperatures, samples for unconfined compressive testing were subjected to different curing temperatures including room temperature (21°C), 35 and 50°C . Samples were put in the oven to cure at constant temperatures of 35 and 50°C . Samples were allowed to cure at these temperatures under both the exposed and sealed conditions.

Delay in Compaction

The cement reacts with water to form cementitious compounds. In general, there are two stages in the process of formation of cementitious material. These are designated as the initial and final setting times of cement. Until the initial setting time, the cement gel behaves like a fluid and it can flow and take any shape with minor changes in its quality. When the final setting time is approached, the gel hardens and the plasticity of the cement is completely lost. These processes are 'irreversible'.

Cement hydration is a time dependent process and the rate of hydration decreases with time. Moreover, in the field, construction of soil-cement involves time consuming processes. These processes include the pulverization of soil to the required degree of fineness, mixing of cement with soil, adding water to bring the moisture content to the desired value, proper mixing of soil-water-cement system, and finally compaction. To ensure a uniform mixing and uniform cement and water content in the treated soil, a time of about 2 to 4 hours is usually required before compaction. On the other hand, the hydration of cement starts immediately upon

mixing with wet soil. A brisk hydration is onset when additional water is added to the mix. This hydration of cement, before compaction, has many consequences.

In order to achieve high strength of the soil-cement mixtures, it is essential to have not only compaction of the mix but also an efficient hydration of the cement. Otherwise, there will be localized weak sections in the compacted soil-cement layers. If the elapsed time between mixing and compaction is more than a certain 'limit', detrimental effects may occur. Delay in compaction causes the cement to hydrate and clods of cemented soil will form. The formation of clods will prevent particle rearrangement during compaction, leading to samples with lower density and therefore lower strength. In addition, the bonds that have been formed will be broken, and less cement will be available as a binding material. This could result in lower strength and durability.

When the compaction of soil and cement mixtures was delayed, after mixing, for certain periods, a considerable loss of strength and durability occurs if the delay is equal to or greater than the initial setting time of cement (Arman and Saifan, 1967). The initial setting time for sulphate resisting cement (Type V) is about three hours. Therefore, delays of 1, 2, and 4 hours were investigated in this work. After the specified periods of delay, the soil-cement mix was compacted to cast samples for the unconfined compressive test. During the delay time, the mix was sealed in plastic bags to prevent any moisture loss. In the field, sealing the treated soil may not be easily achieved. However, there is a practice of adding some extra water to the mix during compaction, to compensate for the moisture loss during the delay (Lightsey et al., 1979). Some of the delayed soil-cement mixtures were

remixed in the mixer for three minutes before being compacted. Sugar (0.5% of the cement weight) was used as a set-retarder for two samples which were delayed for one hour. After compaction, the samples were sealed in plastic sheets and cured for seven days.

3.4.3 Unconfined Compressive Strength

Unconfined compressive strength test is commonly used for the evaluation of chemically-stabilized soils. In the structural design of pavements, the unconfined compressive strength (UCS) is used as the strength criterion for the base and the sub-base layers. However, its use for pavement structure is getting less popular after the adoption of the resilient modulus test, which relates more closely the response of pavement material to impulse loading. However, the UCS is still in use in many standards and codes for the stabilized materials. Usually, a minimum UCS value is specified for different layers. It was therefore adopted as a basic test in this research where a comparative study of the effects of different parameters such as curing regimes, delay in compaction, curing period on the strength gain of the treated marls are to be investigated.

The unconfined compressive strength test (UCT) was also used for evaluating the strength characteristics of both untreated and soil-cement mixtures. Unconfined compressive test is performed on cylindrical specimens with a height to diameter ratio (h/d) of 2. According to ASTM D 559, the soil-cement specimens for unconfined compressive testing can have either h/d of 1.15 or 2.0. Specimens with h/d of 2.0 gives a better indication of the strength as it minimizes the complex stresses generated at the end plates. For specimens having h/d of 1.15, these

stresses interfere with the true strength behavior. The strength of the specimens having h/d of 1.15 is normally greater than those with h/d of 2.0. The ASTM standards provide a multiplication factor for the strength of specimens with h/d ratio in the range of 1.15 to 2.0.

In this research, specimens with h/d ratio of 2.0 were prepared for all unconfined compressive testing. All samples were reconstituted to meet the gradation requirement. The required cement and the corresponding (optimum) moisture were added to the marl and mixed in a mechanical mixer for 4 min. Then, the mix was compacted in a mold of 100 mm diameter and 200 mm height ($h/d=2.0$) to the maximum dry density of the treated marl according to the Modified Proctor compaction test. The mix was compacted in 5 equal layers. The number of blows were adjusted to attain the maximum dry density at the optimum moisture content. The mold used is of a split type with a longitudinal slit along its axis. The slit is tightened and opened with the help of bolts. After compaction, the specimen was taken out of the mold by loosening the bolts.

3.4.4 Resilient Modulus

The resilient modulus M_R is a dynamic test response defined as the ratio of the repeated axial deviator stress σ_d to the recoverable axial strain ϵ_a (Yoder and Witczak, 1975).

$$M_R = \sigma_d / \epsilon_a \quad (3.1)$$

Since the inclusion of resilient modulus test in the new AASHTO design guide for pavements (AASHTO Design Guide, 1986) for the characterization of material properties, it has started replacing the static strength tests. The resilient modulus

test is dynamic in nature, and simulates the response of the pavement layers to the impulse loading produced by the moving wheels of vehicles. Moreover, the conventional strength tests determine only the relative suitability of the material based on their "static" strength, while properly designed pavement layers seldom receive loads that are close to failure loads. The performance of a material at low impulse load levels is quite different from that at higher load levels.

The impulse load causes deformation in pavement layers consisting of plastic and elastic components. The elastic deformation is termed recoverable or resilient. Before 1986, studies of resilient modulus of various types of aggregates and soils were mainly motivated by the fact that the failure of highway pavement could result not only from excessive accumulated permanent deformation of subgrade under repeated traffic loading, but also from the fatigue cracking of the asphalt concrete surface caused by the repeated resilient deformation (Li and Selig, 1994). Later on, the resilient modulus study of subgrade soils and other pavements layers has become essential in pavement design procedures.

In this investigation, the test adopted for stabilized material was the diametral resilient modulus type. Stabilized soil samples of 101.6 mm (4 in.) in diameter and 63.5 mm (2.5 in.) in length were prepared. The samples were compacted in one layer. Blows were adjusted to achieve the required maximum dry density based on the Modified Proctor test on the treated soil. After compaction, the samples were extruded from the mold and subjected to different curing regimes similar to those for the unconfined compressive test. At the end of curing period, these samples were exposed to diametral resilient modulus testing.

For testing, each stabilized sample was fixed in the testing mold. The mold containing the sample was fixed in position in the resilient modulus device, by applying a seating stress of 4 psi (27.6 kPa). The mold contains two LVDTs on the two opposite ends of the sample. LVDTs were used to measure the diametrical resilient tensile strain produced upon the application of impulse load. An impulse load of 150 lb. (68.2 kg) was applied normal to the direction of installed LVDTs. The resilient strain at the end of the 100th cycle of impulse load was used in calculating the resilient modulus (M_R) of the stabilized soil. Both the applied load and the resulting strain were automatically measured by electronic devices and transferred to a resilient modulus computing system.

3.4.5 Durability

Stabilized soils need to be strong and durable so as to resist physical loads under different exposure conditions. Moisture combined with temperature, can produce wet and dry or freeze and thaw cycles. The stabilized soils should maintain stability under the cyclic environmental loadings. These conditions cause weight loss and/or volume change to the soil-cement. The volume change creates stresses in the soil-cement and contributes directly to the loss of serviceability and strength.

Durability of soil-cement specimens was evaluated using two different procedures, the standard ASTM D 559 method and the proposed slake durability test (Goodman, 1980). The latter is originally used for rocks but modified to accommodate cement-stabilized soils with certain sizes.

Standard Durability Test (ASTM D 559)

Soil-cement specimens were prepared with different percentages (3, 5 and 7%) of cement. The molds used to prepare specimens were 101.6 mm (4 in.) in diameter and 116.8 mm (4.6 in.) in height. Each sample was compacted to the modified Proctor maximum density of the treated soil. The number of blows was adjusted and the sample was compacted in three layers. After compaction, the sample was extruded from the mold. Four samples were prepared for each cement content. Two of these samples were designated as weight loss samples, while the other two were designated as volume change samples. The height and diameter of the volume change samples were measured.

All samples were cured under 100 % relative humidity and a temperature of $21 \pm 2^\circ\text{C}$ for 7 days. Thereafter, the samples were placed in a water tank for 5 hours at room temperature and thereafter transferred to an oven. Samples were kept in the oven at 71°C for 42 hours. This is what constitutes one wet-dry cycle for the soil-cement samples.

At the end of this cycle, the samples designated as volume change samples were dimensioned with the help of a vernier caliper, and were weighed. Using a standard brush, the other two samples were brushed with two strokes on the whole surface with a force of about 3 lb. (1.36 kg). To apply the 3 lb. (1.36 kg) force, each sample was placed on a balance, and was then brushed while observing the specified force on the scale of the balance. The weight of the samples before and after brushing was measured.

Similar procedures were repeated for the remaining 11 cycles, thus subjecting the samples to a total of 12 cycles. At the end of each cycle, the weight loss and volume change were noted for the respective samples. At the end of the 12th cycle, the samples were dried to constant weight at 110°C.

Slake Durability Test

This test is based on the slake durability of rocks (Goodman, 1980). A certain specific weight (500 gm) of rock pieces is put in a drum made of 2.00 mm mesh. The drum is 100 mm in length and 140 mm in diameter. The drum is rotated at a speed of 20 rpm while being partially submerged in water. The weight loss after 10 min. of rotation, is a measure of the durability of the rock.

The above test was adopted for the soil-cement samples with some modifications. The diameter and length of the drum was changed to 152.4 mm (6 in.) long and 304.8 mm (12 in.) in diameter. To allow the soil-cement specimens to travel the same distance as by the rock pieces in the original test, the number of revolutions was adjusted to account for the change in dimensions. The revolution time was reduced from 10 to 4.6 min. This will give a total travel distance of 88 m. The setup for slake durability testing is shown in Fig. 3.5.

Two additional samples were compacted for each percentage of cement. These samples were subjected to the same wet and dry cycles as for the samples tested using ASTM D 559 durability test. At the end of each cycle, each sample was tested for slake durability using the modified drum. After slaking, the surface of the sample was cleaned with a dry absorbent cloth and then weighed. For each

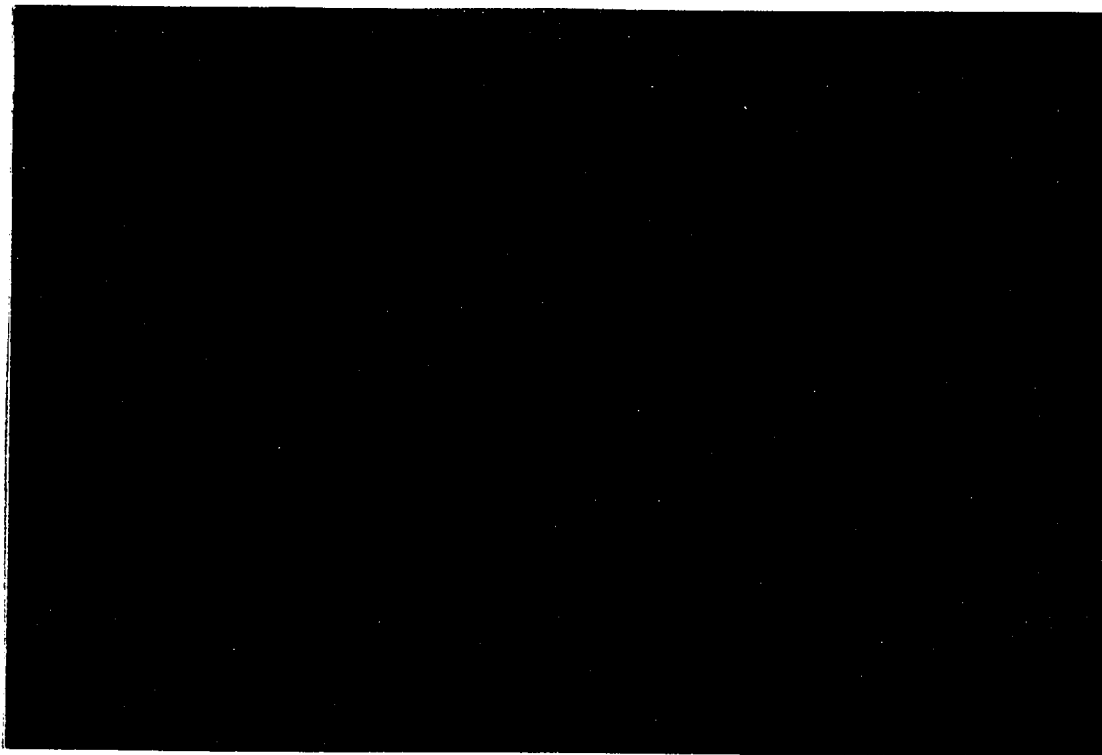


Figure 3.5: Modified Apparatus for Slake Durability Testing of Soil-Cement Specimens (Aiban et al., 1995b).

cycle, weight loss for each sample was noted by taking the weight before and after the slaking test. After 12 cycles, the samples were oven dried at 110°C.

3.5 Grain Crushing

The engineering properties of soils, such as strength, compressibility, and permeability, depend to a certain degree on their gradation. Gradation is a basic parameter in the classification systems of soils, which in turn is used to predict the behavior of soils. A specific grain-size distribution (GSD) is always recommended when the soil is to be used as a fill material. The natural gradation is modified, if needed, to conform to the required gradation. Fig. 3.6 shows the ASTM grain-size limits (ASTM D1241), four gradations are specified for Type I material, for the use in base course construction. Gradation A is the coarsest, while D is the finest of these four gradations. According to this ASTM standard, Type I materials are the mixtures consisting of stone, gravel, or slag; with natural or crushed sand and fine mineral particles passing No. 200 sieve.

When soils are compacted, some of the grains are expected to be crushed. The degree of crushing is highly dependent on the type of material and the compaction technique. Soils containing soft carbonate aggregates are more susceptible to crushing during compaction when compared to non-carbonate soils. During construction of a fill, the required density is achieved using certain compaction techniques and efforts. The compaction energy causes the breakdown of the soft aggregates present in the soil and, consequently, the gradation of the soil after compaction in the field becomes different from what has been specified. The specified gradation must be met in the field so that the compacted layer will behave

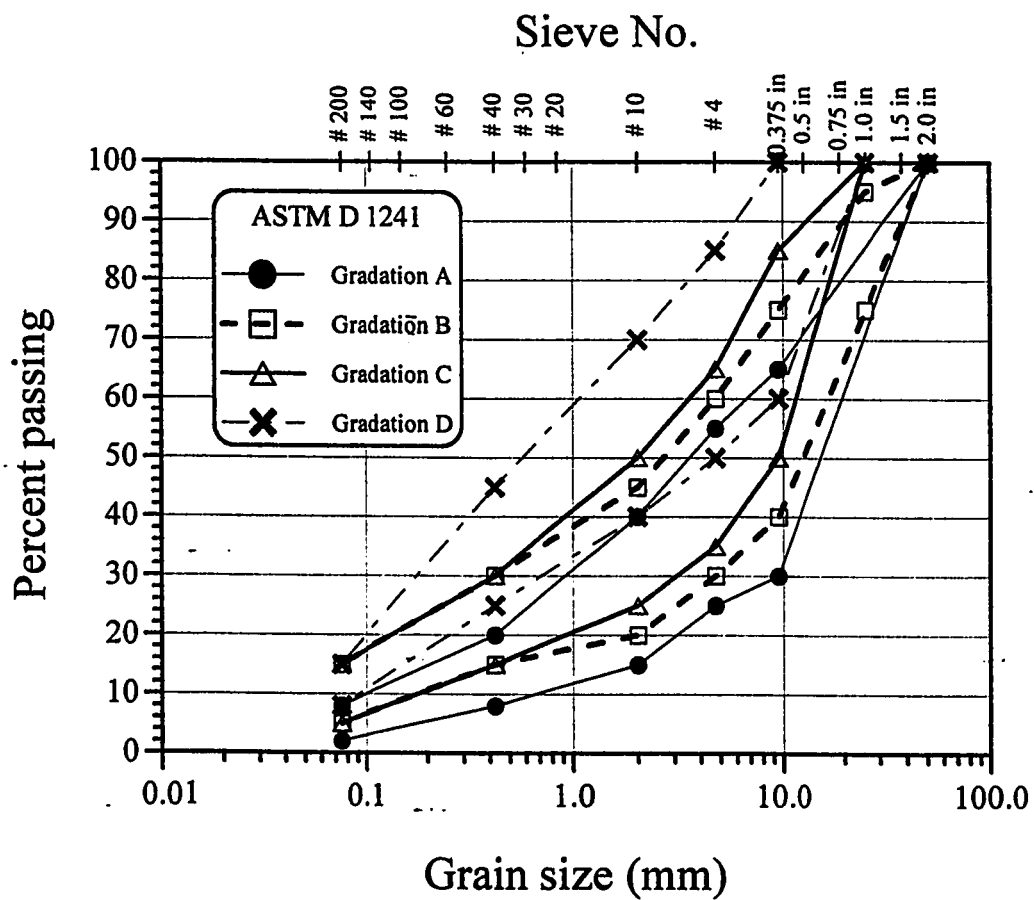


Figure 3.6: Specified Limits of Different Gradations for Base Course (ASTM D1241).

satisfactorily. Laboratory compaction techniques were used to investigate the effect of different compaction methods on the crushing of marl soils.

The effect of laboratory compaction methods on the crushing of soil particles was investigated for one of the selected marls. The compaction methods adopted were the standard Proctor, the modified Proctor, and the static compaction. The static compaction was performed using a hydraulic compression machine with a loading rate of 1.25 mm/minute. The first two methods are considered to be dynamic in nature, while the last one is static. The samples used were 152.4 mm (6.0 in) in diameter and 127.0 mm (5.0 in) in height, and were compacted in five layers.

The samples were reconstituted to the dry gradation curve and were compacted at the optimum moisture content, as well as on dry and wet sides of the optimum, corresponding to 95% of the maximum dry density. After compaction, the samples were extruded from the molds, soaked for 48 hours, and then subjected to wet sieve analysis. The grain-size distribution curves before and after the compaction were plotted for each sample for relative comparison.

Chapter 4

Results and Discussion

4.1 Characteristics of Marl Sites

Marl samples were collected from 24 different sites distributed all over the eastern Saudi Arabia. Sampling areas include Abu Hadriyah, Baggah, Hofuf, Abqaiq, Dhahran, Shedgum, and Ain Dar. Most of these sites are being used as borrow areas by contractors; however, some of these sites had almost been depleted because of the extensive use in the construction of infrastructure for almost the last 20 years. Some of the collected samples were reconstituted to a specific base course gradation, while the others were reconstituted to their natural gradation.

Soil is heterogeneous in nature both in the horizontal as well as in the vertical directions. The variability exists in the thickness of different layers as well as in the constituents of the individual layers. The obvious reason is that the formation of rock and soil is caused by different geological processes which are continuously varying. The processes of weathering, transportation and deposition of the sediments depend on various environmental and physical factors, which are never the same.

The eastern part of the Arabian Peninsula is inherited by different geological formations. It consists of Precambrian sedimentary and volcanic rocks which subsequently got metamorphosed and have intruded plutonic formations (Al-Sayari and Zotl, 1978). It is overlain by Paleozoic, Mesozoic, and early Cenozoic sedimentary strata. The surface formations include consolidated sedimentary formations ranging from Paleocene to Middle Eocene age and Miocene to Pliocene age (Al-Sayari and Zotl, 1978). Unconsolidated materials of Tertiary age and sediments of Quaternary age are also present. The surface formations resulted from the sedimentary shelf sea environment during which the shoreline suffered a large back and forth movements.

These surface formations show wide variations in layering as well as in mineralogy. Therefore, the samples collected from different sites within the Eastern Province are only representative of the particular sites of collection at that time. Each sample should not even be considered representative of the whole width and depth of the borrow area.

The marl samples collected from Abu Hadriyah, Hofuf, Abqaiq, Baggah, Dhahran, Shedgum, and Ain Dar areas are located in specific geological formations as shown in Fig. 4.1. The surface deposits or formations in the Eastern Province belong to different geological periods. The soils from these formations contain specific minerals reflecting these formations. These minerals, in turn, reflect the engineering behavior of the deposits. Calcite and dolomite are the dominant minerals existing in these soils. Calcite is the primary mineral deposited during the formation of

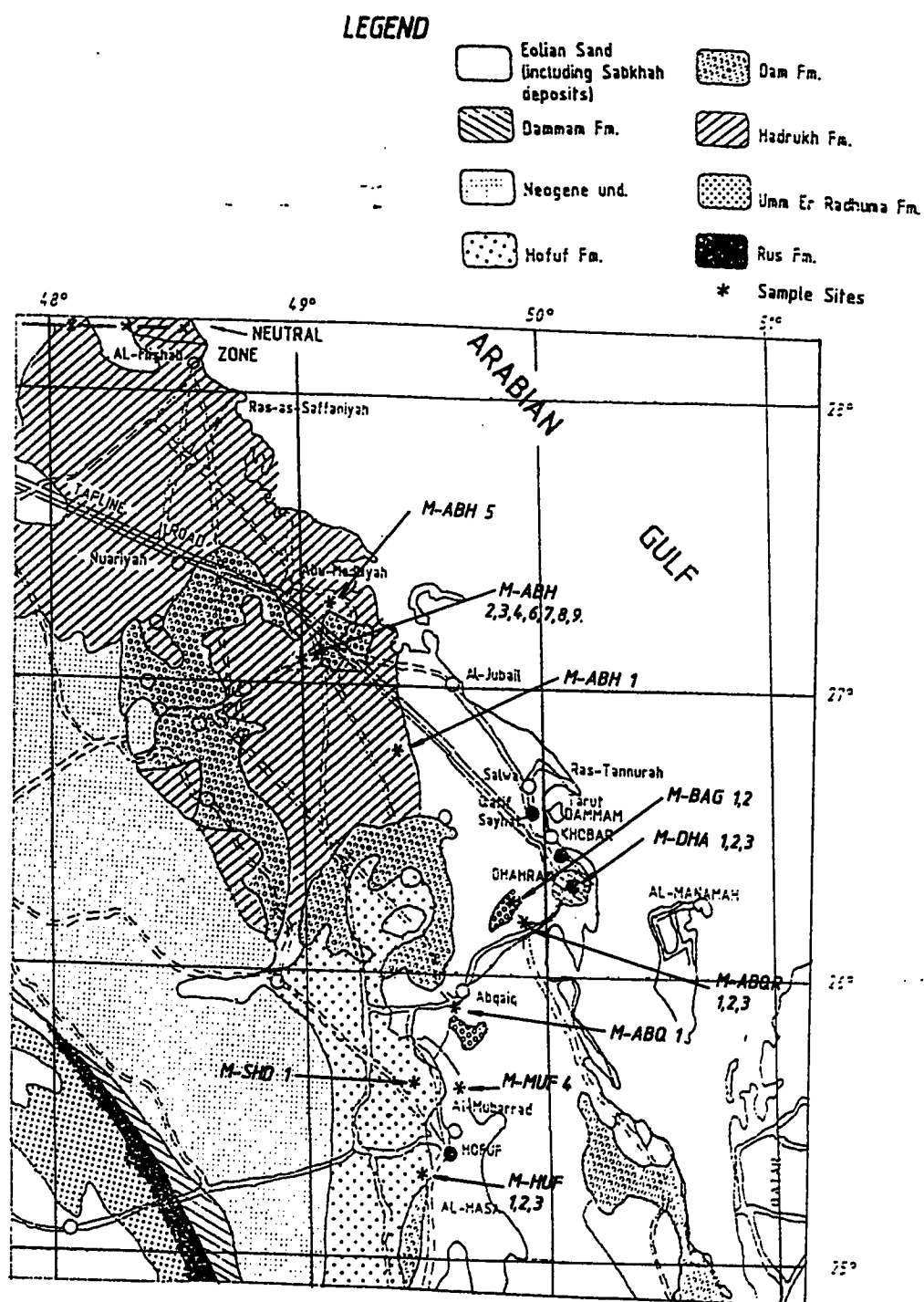


Figure 4.1: Geologic Formations for the Different Locations of the Collected Marl Sites.

sedimentary calcareous soils. Later on, it might be dolomitized to form dolomite and dolomitic limestone. The formation of minerals such as dolomite, chert, flint, jasper, gypsum, anhydrite, etc., results from processes collectively termed as 'metasomatism' (Blyth and deFreitas, 1985). Metasomatism has been explained in the context of formation of sedimentary calcareous soils. Quartz geodes of different sizes also exist in the collected soil samples. The existence of geodes is also a characteristic of calcareous soils. They are hollow, potato-shaped nodules having a chalcedonic lining, showing the projecting calcite crystals inside the cavity. Some of the formations also contain shale layers (Al-Sayari and Zotl, 1978). These resulted either from the transformation of certain minerals into clay under certain environmental conditions or from the sedimentation of the clay mud particles in stillwater lagoons.

Marl has been defined in several different ways in the context of sedimentary calcareous soils. - The common part of all definitions is the presence of a small portion of clay besides calcium carbonate. The soil samples collected from the different sites has been termed 'marl' so far. The mineralogical composition and the behavior of these soil samples showed the absence of clay minerals in most of these samples, while only traces ($<3\%$) of clay minerals are present in few others as shown in Table 4.1. Geological evidence shows the presence of marl rock in the area (Table 2.3), however, these are present in certain formations at certain depths. Therefore, care should be taken while naming the soils or rocks in the area.

Based on the definition of marl and the mineralogy of the collected soil samples, labeling them as 'marl' could be misleading. These may be named according to

Table 4.1: Mineralogical Composition of Eastern Saudi Marl Soils from XRD Analysis

Marl symbol	Calcite (%)		Dolomite (%)		Quartz (%)		Others [§] (%)	
	-* #40	- #100	- #40	- #100	- #40	- #100	- #40	- #100
M-ABH1	73.00	90.00	---	---	27.00	10.00	---	---
M-ABH2	96.00	80.00	1.00	1.00	3.00	18.00	---	1.00
M-ABH3	31.00	40.00	66.00	55.00	3.00	5.00	---	---
M-ABH4	65.00	29.00	31.00	67.00	3.00	4.00	---	---
M-ABH5	29.00	15.00	67.00	83.00	4.00	2.00	---	---
M-ABH6	54.00	70.00	11.00	13.00	35.00	16.00	---	1.00
M-ABH7	70.00	4.00	13.00	87.00	16.00	3.00	1.00	6.00
M-ABH9	96.00	80.00	1.00	1.00	3.00	18.00	---	1.00
M-ABQ1	16.00	12.00	34.00	47.00	48.00	38.00	2.00	3.00
M-ABQR1	9.00	---	63.00	43.00	28.00	57.00	---	---
M-ABQR3	5.00	13.00	20.00	38.00	72.00	45.00	3.00	4.00
M-AIND	---	---	81.00	78.00	13.00	13.00	25.00	9.00
M-BAG1	---	---	75.00	79.00	16.00	13.00	4.00	9.00
M-BAG2	6.00	19.00	75.00	35.00	---	17.00	19.00	29.00
M-DHA1	3.00	3.00	97.00	97.00	---	5.00	---	5.00
M-SHD1	28.00	71.00	---	---	70.00	23.00	2.00	6.00

[§] include illite, sepolite, montmorillonite, palygorskite and gypsum

* - implies percent passing sieve #

the minerals present in the particular soil samples. Some of these contain calcium carbonate as the main constituent, while dolomite is present in other as the main mineral. Since 'carbonate' is their main constituent, these soils can be termed 'carbonate soils'. Other alternative names can be 'calcareous soil', 'calcitic soil', or 'dolomitic soil' depending on the type of main mineral present in the soil. 'Calcareous' has been defined as 'pertinent to calcium carbonate or limestone' (Challinor, 1978). Therefore, the term 'calcareous' cannot be used for the soil rich in dolomite mineral. In that case, 'dolomitic soil' will be an appropriate term. However, the term 'marl' will continue to be used in this thesis for the calcareous soils found in the Eastern Province of Saudi Arabia since people have adopted it.

4.2 Characterization of Eastern Saudi Marls

4.2.1 Classification of Marl

The engineering classification of soils provides an idea about their engineering behavior. In the AASHTO classification system, the A-1-a and A-1-b soils are considered the best materials for use in pavement structures. Similarly, in the Unified Soil Classification System (USCS), the GW, SW soils are equivalent to A-1 soils (Holtz and Kovacs, 1981). It has been observed that the marl soils obtained from the Eastern Province are not sensitive to the above mentioned classification systems. Similarly, plastic marls, which are usually considered to be unsuitable materials for use in construction fills, are the ones that can be regarded satisfactory.

Laboratory tests were performed on marl samples, collected from different sites, to characterize these soils. The plasticity and grain-size distribution were used to classify these soils. The soils were classified according to AASHTO and Unified

Soil Classification Systems (USCS). The classification and some other relevant engineering properties are shown in Tables 4.2 and 4.3. A large difference exists in case of dry and wet sieving. In the presence of some moisture, the fines form aggregations which are responsible for the apparent coarse gradation during dry sieving. These aggregations break and transform to individual particles during wet sieving. Besides being in the form of lumps, the fines tend to coat the larger particles. On the other hand, the wet sieving provides a clear picture of all particle sizes as well as their percentages. Moreover, the fines obtained in the wet sieving are used for hydrometer analysis as these are truly representative of the particle sizes less than $75\ \mu\text{m}$. In the case of soils having large percentage of clayey particles, wet sieving is considered to be a representative. However, carbonate particles are susceptible to dissolution in water. Therefore, the wet sieving may not give accurate results for these soils.

4.2.2 Strength and Compactibility

In addition to the basic characterization tests, modified Proctor compaction tests, and CBR tests were performed, to evaluate the suitability of these soils as construction materials. These tests indicated the deficiencies of these soils, and the possible improvements that can be made for their utilization in different projects. The variation of density and CBR with the molding moisture content was plotted for the collected marl samples in Figs. 4.2 to 4.25. The general and particular observations regarding the characterization of marl soils are discussed in the following sections.

Table 4.2: Index Properties and Classification of Marl Soils

Marl Symbol	Specific Gravity	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% passing #200 sieve		Classification	
		^s #40 / - #100	- #40 / - #100	- #40 / - #100	Dry	Washed	USCS	AASHTO
M-ABH1	2.84	ND / 21.5	ND / 20.0	ND / 1.5	10.72	29.57	SP-SM	A-1-b
M-ABH2	2.73	17.0 / 21.0	14.3 / 19.5	2.7 / 1.5	3.44	26.5	SP	A-1-b
M-ABH3	2.80	Non-plastic			1.01	22.95	SP	A-1-a
M-ABH4	2.80	Non-plastic			0.22	9.56	GP	A-1-a
M-ABH5	2.80	Non-plastic			4.78	18.51	GW	A-1-a
M-ABH6	2.73	Non-plastic			7.45	18.14	SP-SM	A-1-b
M-ABH7	2.81	36.3 / 38.4	28.3 / 32.5	8.0 / 5.9	4.19	44.37	SP	A-2-4
M-ABH8	2.81	Non-plastic			1.92	24.71	SP	A-1-a
M-ABH9	2.73	17.0 / 21.0	14.3 / 19.5	2.7 / 1.5	2	12.5	GP	A-1-a
M-ABQ1	2.71	47.0 / ND	45.0 / 48.7	2.0 / 2.4	18.94	22.9	GM	A-1-a
M-ABQR1	2.72	85.0 / 99.7	61.4 / 66.7	23.6 / 33.0	7.21	17.31	GP-GM	A-2-7
M-ABQR2	2.85	Non-plastic			9.63	27.49	SP-SM	A-1-b
M-ABQR3	2.73	33.4 / 34.9	26.1 / 30.0	7.3 / 4.9	0.93	18.72	GP	A-2-4
M-AIND	2.78	68.3 / 72.5	64.7 / 69.4	3.6 / 3.1	9.54	22.75	SP-SM	A-1-a
M-BAG1	2.92	95.2 / 105.0	66.4 / 82.7	30.3 / 24.2	7.31	27.12	SW-SM	A-2-7
M-BAG2	2.66	89.3 / 123.1	64.9 / 90.0	24.4 / 33.1	8.41	27.32	SP-SM	A-2-7
M-DHA1	2.91	36.5 / 40.7	32.3 / 35.8	4.2 / 4.9	18.16	30.45	GM	A-1-b
M-DHA2	2.74*	107.7* / 89.7*	63.5* / 60.9*	44.2* / 28.8*	27.5	39.26	ND	ND
M-DHA3	2.85	ND / 34.2	ND / 20.3	ND / 13.9	11.18	22.12	GP-GM	A-1-a
M-HOF1	2.70	Non-plastic			1.5	9.9	GP	A-1-a
M-HOF2	2.75	32.3 / 46.8	22.9 / 25.2	9.4 / 21.6	5	21.64	SP	A-2-4
M-HOF3	2.84	86.6 / 84.4	47.5 / 42.4	39.1 / 42.0	3	29.35	GW	A-1-a
M-HOF4	2.86	Non-plastic			3	40.09	SP	A-1-b
M-SHD1	2.69	Non-plastic			2.55	19.23	SP	A-1-a

ND : not defined

* for clayey material only

^s - implies percent passing sieve #

Table 4.3: Compaction and CBR Characteristics of Marl Soils

Marl symbol	Max. Dry Density (γ_d) _{max} (g/cm ³)	Optimum Moisture Content (%)	CBR			Maximum CBR	
			at 0.95(γ_d) _{max} (dry side)	at (γ_d) _{max}	at 0.95(γ_d) _{max} (wet side)	CBR	OMC _{CBR}
M-ABH1	1.85	13.00	110.0	104.4	0.0	130.0	11.0
M-ABH2	2.09	7.25	100.0	130.0	0.0	140.0	6.8
M-ABH3	1.93	10.25	ND	130.0	ND	130.0	9.0
M-ABH4	2.15*	8.00**	ND	180	ND	180.0	8.0
M-ABH5	1.91	14.10	ND	122.0	2.0	152.0	12.9
M-ABH6	2.17	6.80	123.0	145.0	10.0	163.0	6.4
M-ABH7	1.80	16.65	ND	79.0	6.0	90.0*	14.5*
M-ABH8	1.93	12.00	ND	135.0	ND	197.0	8.0
M-ABH9	2.20	6.40	115.0	135.0	10.0	135.0	6.4
M-ABQ1	1.85	14.00	90.0	155.0	ND	155.0	14.0
M-ABQR1	1.81	15.00	ND	180.0	50.0	180.0	15.0
M-ABQR2	1.79	14.90	88.8	148.4	35.0	155.0	13.5
M-ABQR3	2.06	8.60	ND	50.0	ND	128.0	7.0
M-AIND	1.87	13.50	70.0	130.0	ND	141.0	12.5
M-BAG1	1.69	18.61	ND	81.0	36.0	82.0	20.9
M-BAG2	1.79	17.50	ND	125.0	40.5	142.0	16.5
M-DHA1	1.75	16.00	100.0	107.0	28.0	108.0	14.0
M-DHA2	1.51	21.84	ND	45.0	20.0	73.0*	17.8.0*
M-DHA3	1.97	12.56	ND	111.9	17.5	250.0	11.0
M-HOF1	2.20	6.78	ND	204.5	ND	204.5	6.8
M-HOF2	2.07	8.50	ND	165.0	8.0	165.0	8.5
M-HOF3	1.67	18.53	ND	87.6	22.0	92.0	20.0
M-HOF4	1.94	12.40	85.0	65.0	10.0	125.0	11.0
M-SHD1	2.11	9.00	ND	160.0	20.0	210.0	7.0

ND : not defined

* no optimum was obtained

** one for maximum CBR

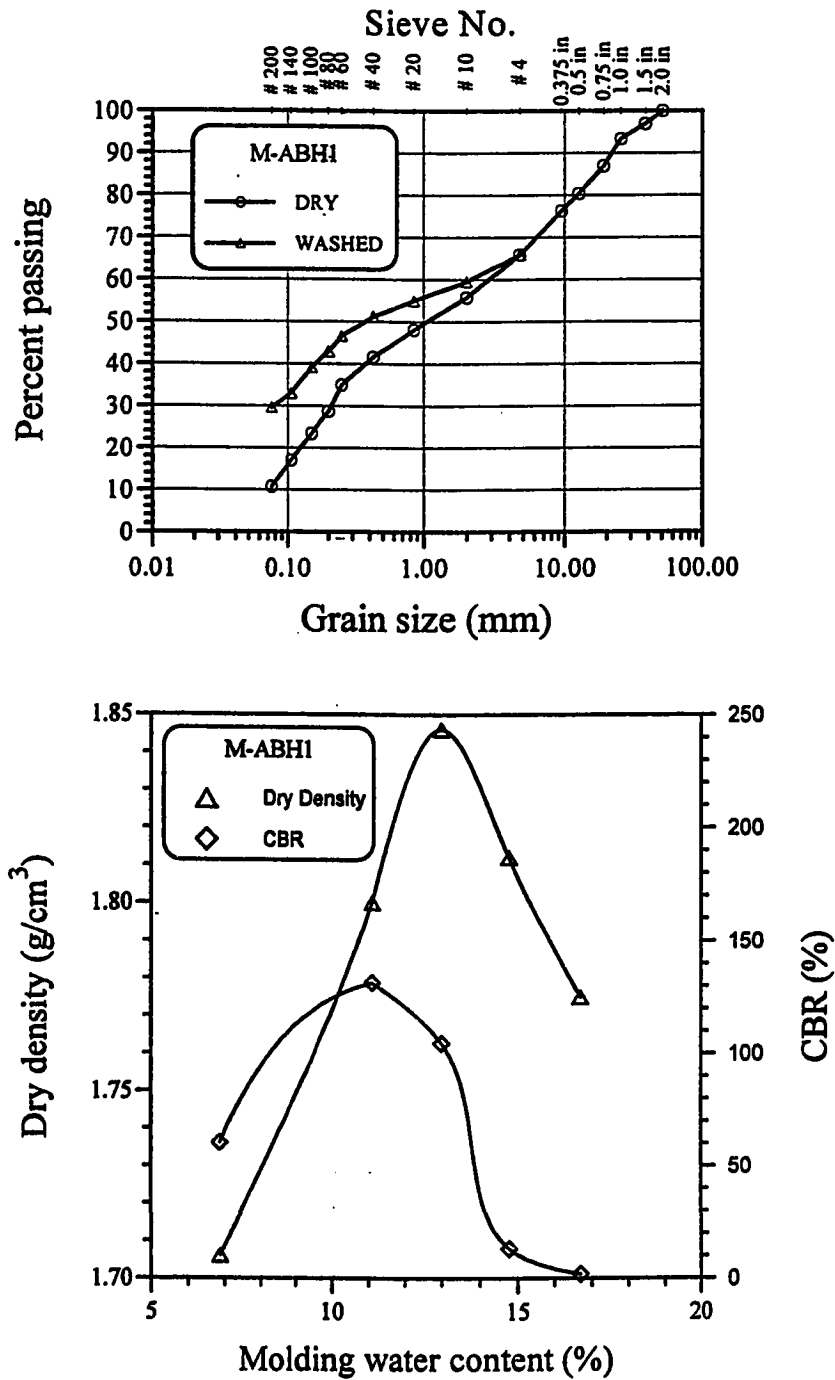


Figure 4.2: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH1.

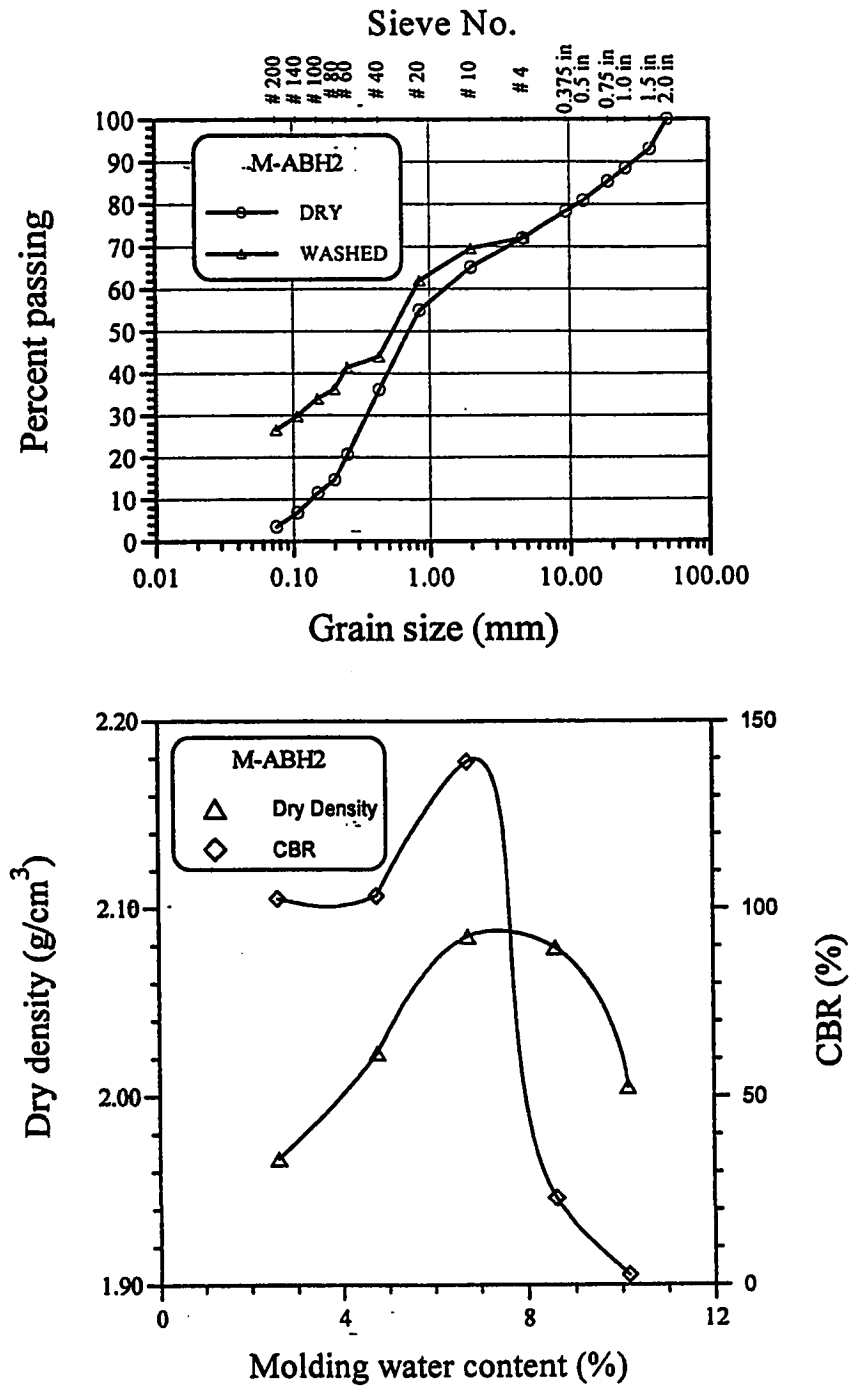


Figure 4.3: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH2.

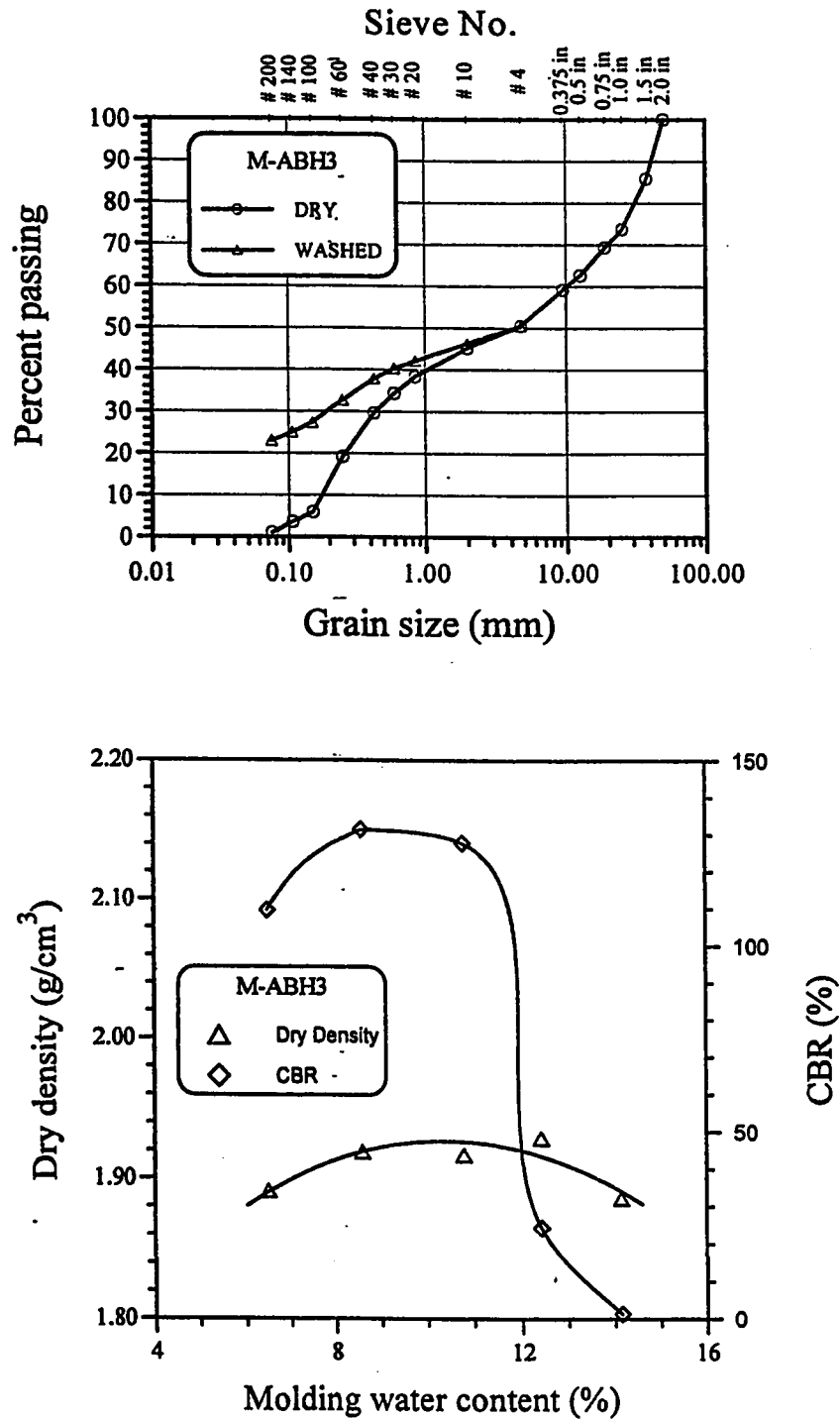


Figure 4.4: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH3.

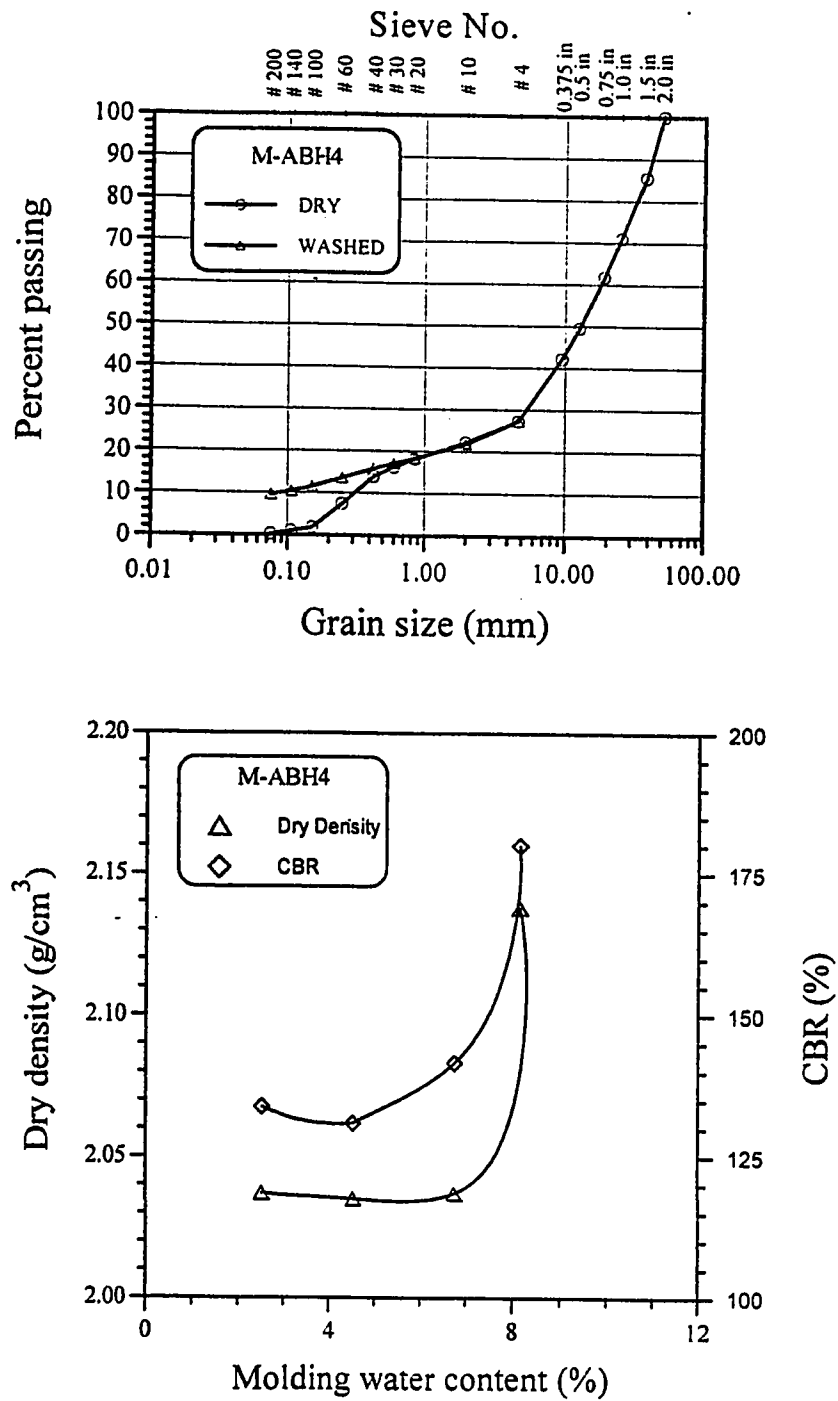


Figure 4.5: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH4.

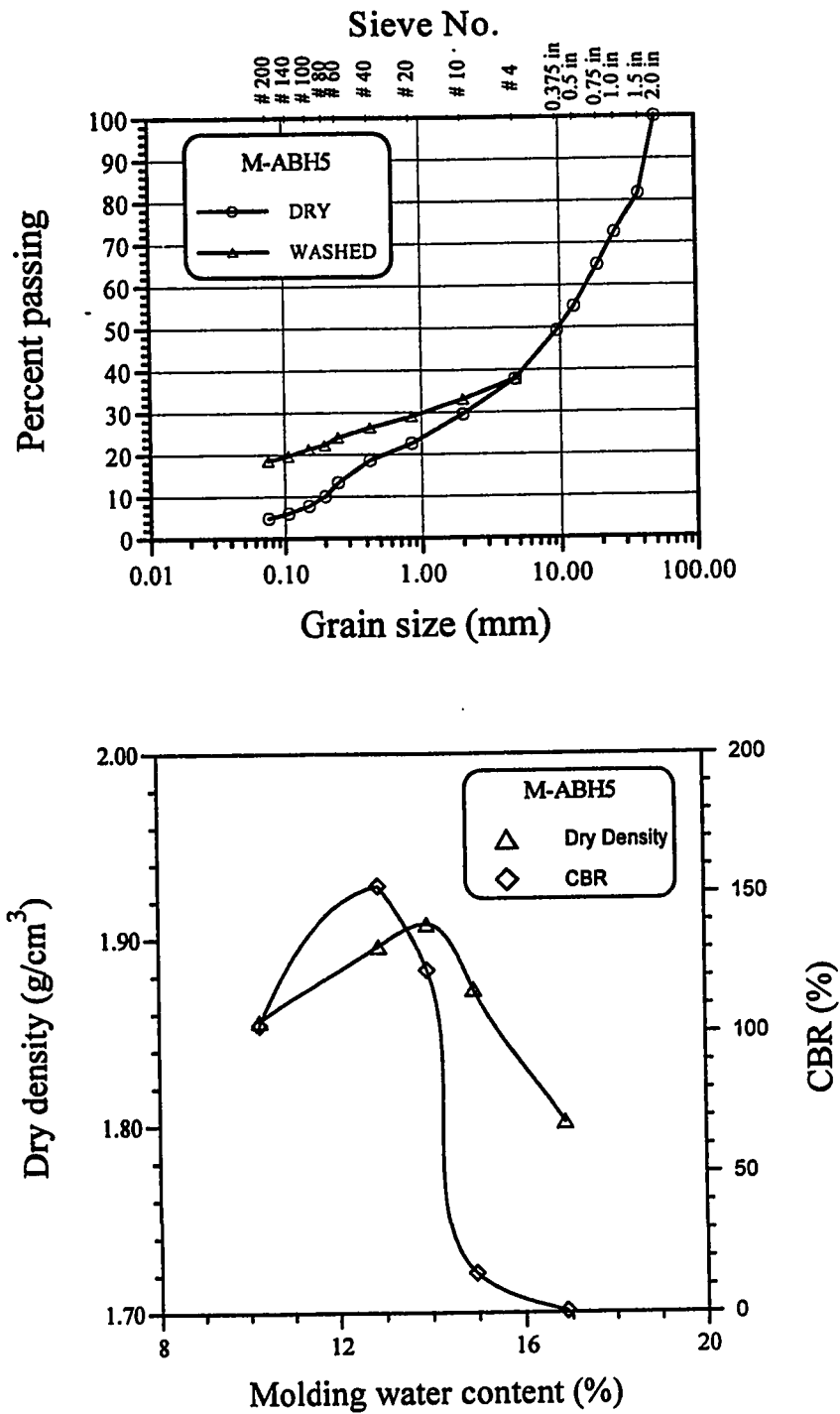


Figure 4.6: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH5.

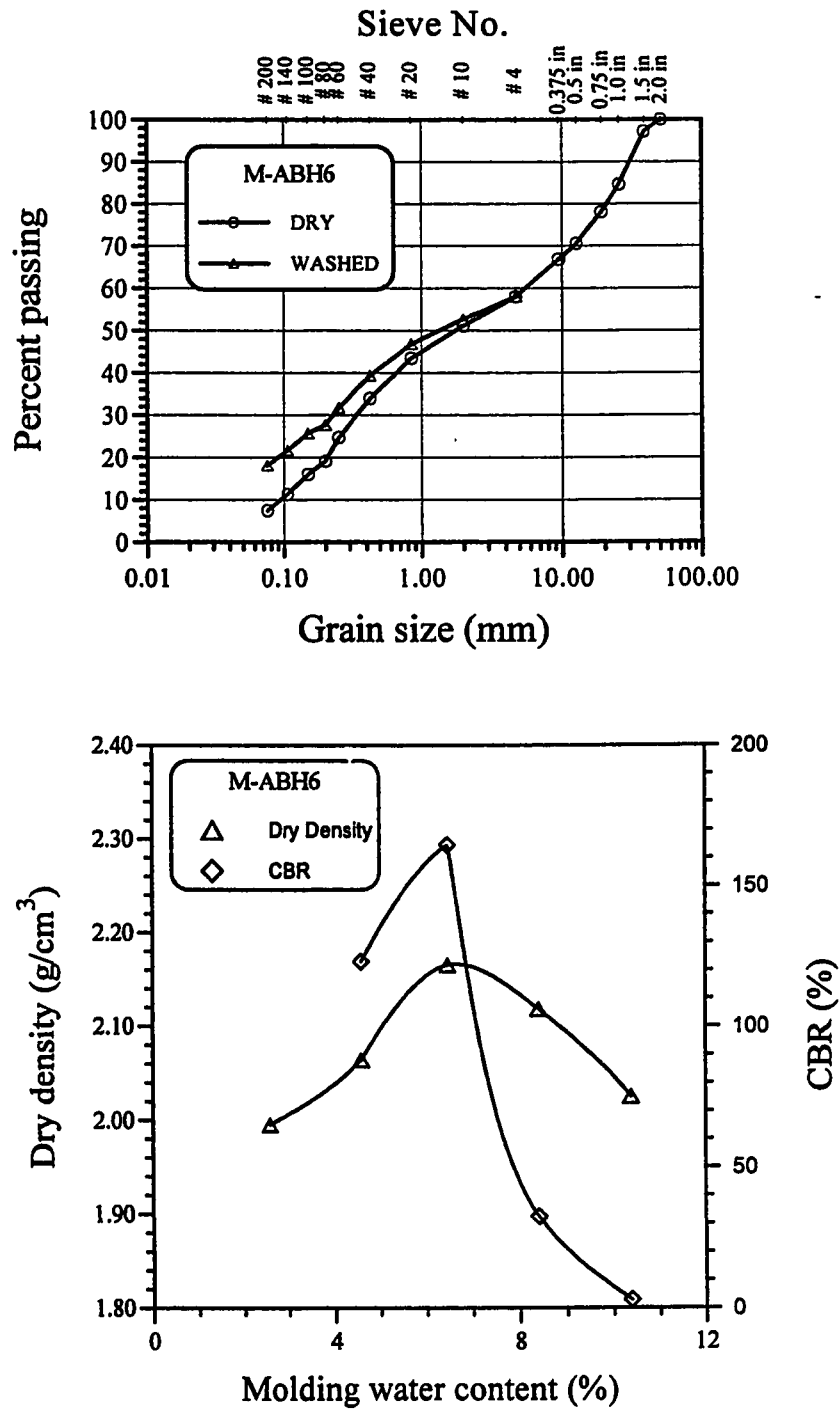


Figure 4.7: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH6.

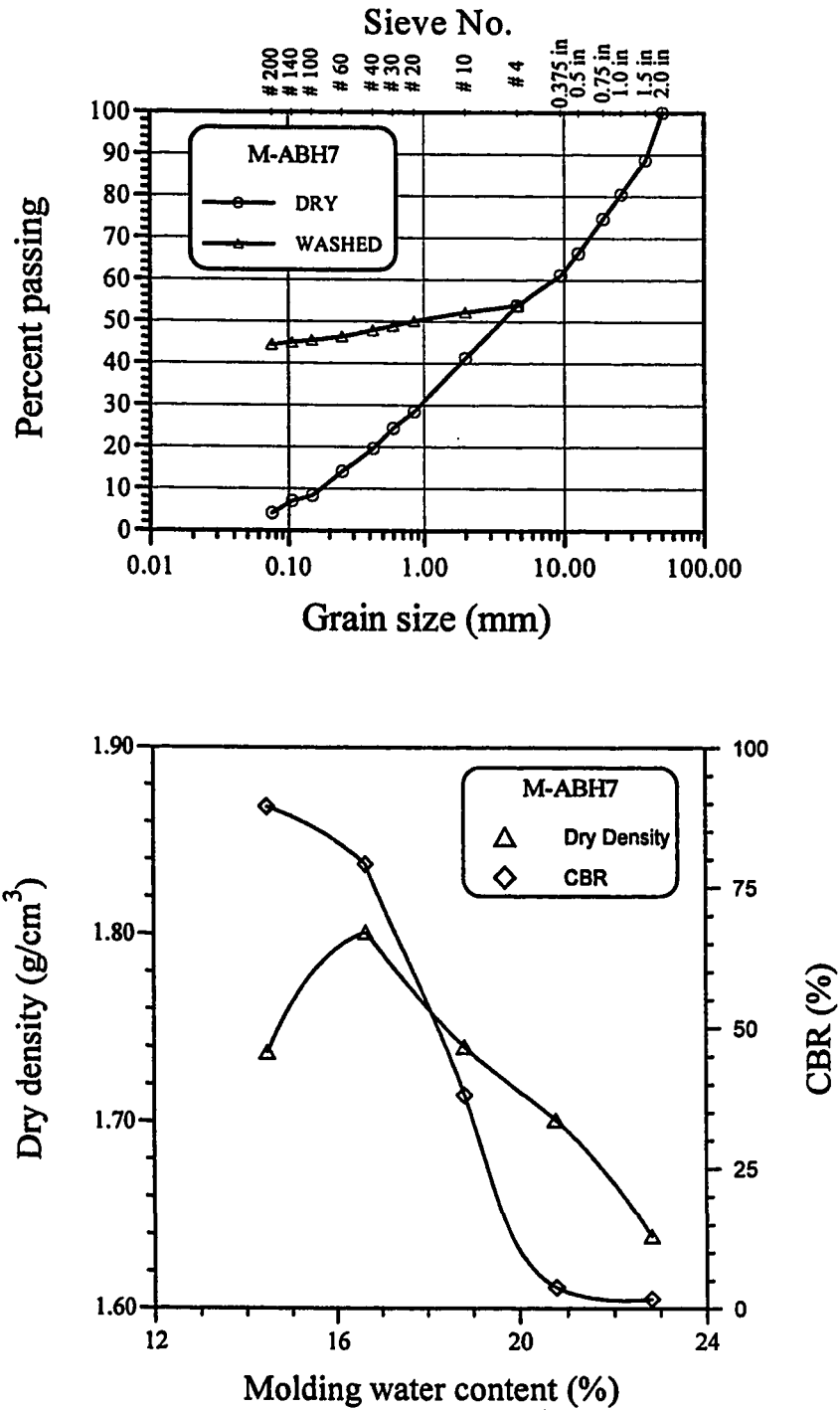


Figure 4.8: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH7.

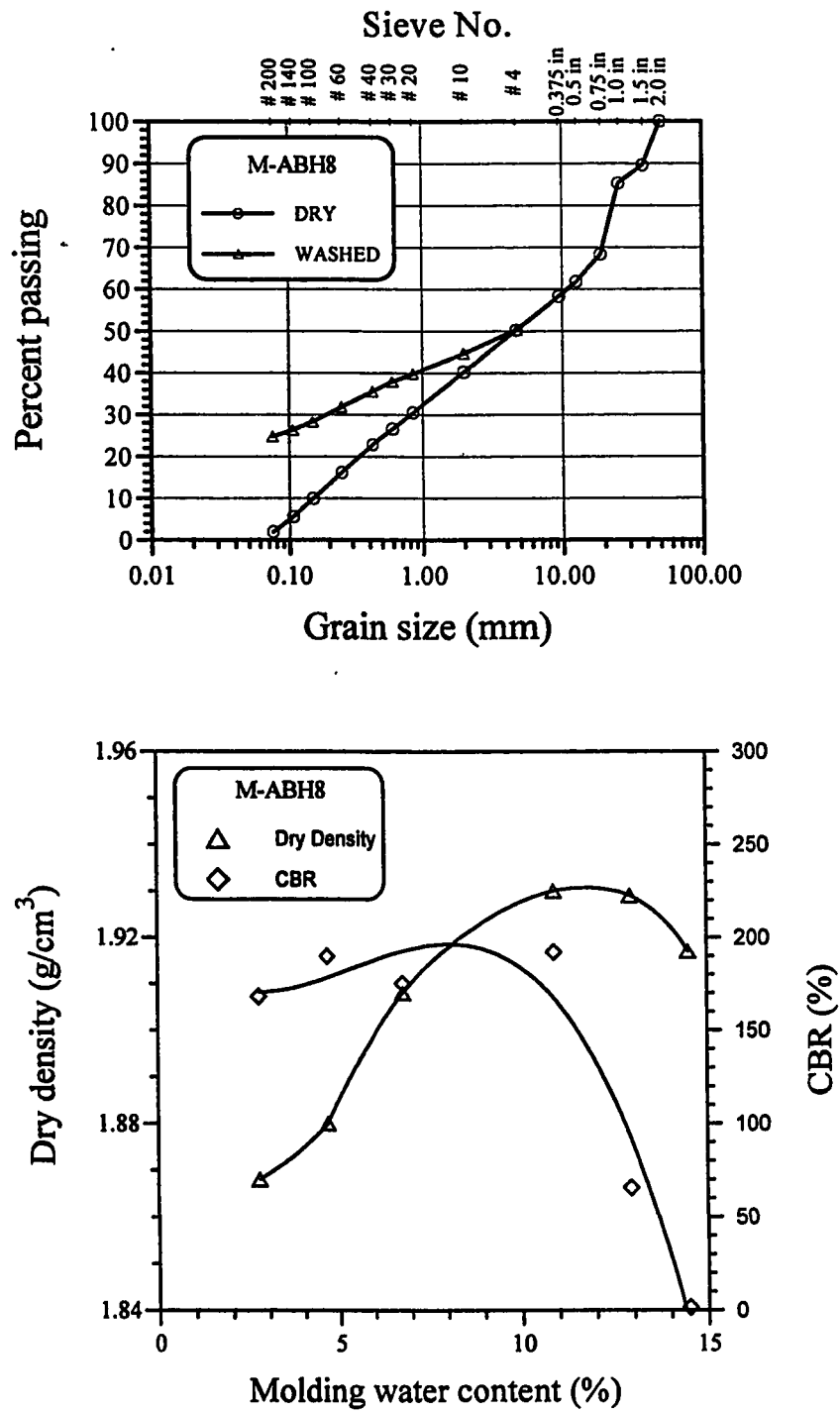


Figure 4.9: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH8.

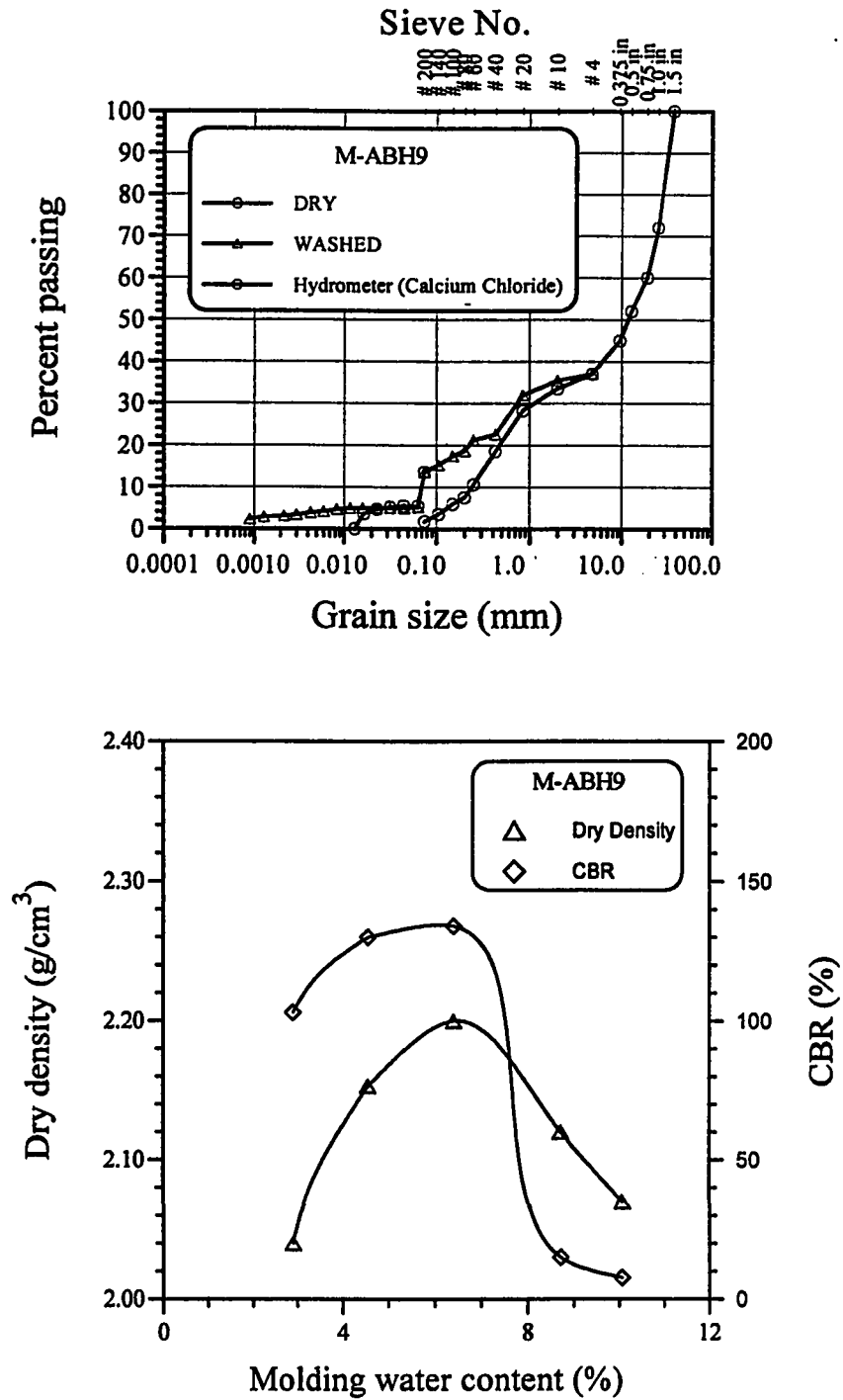


Figure 4.10: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABH9.

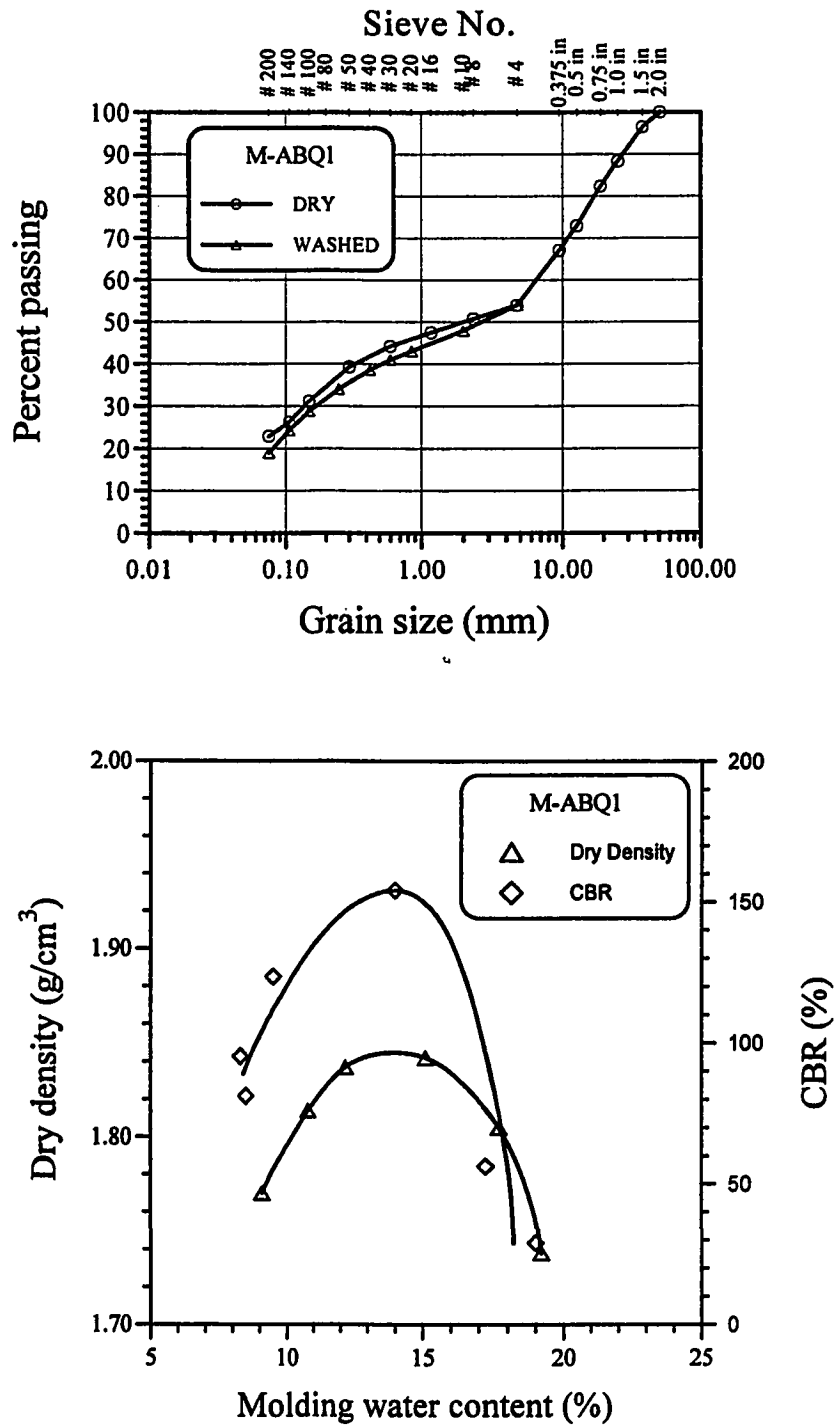


Figure 4.11: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABQ1.

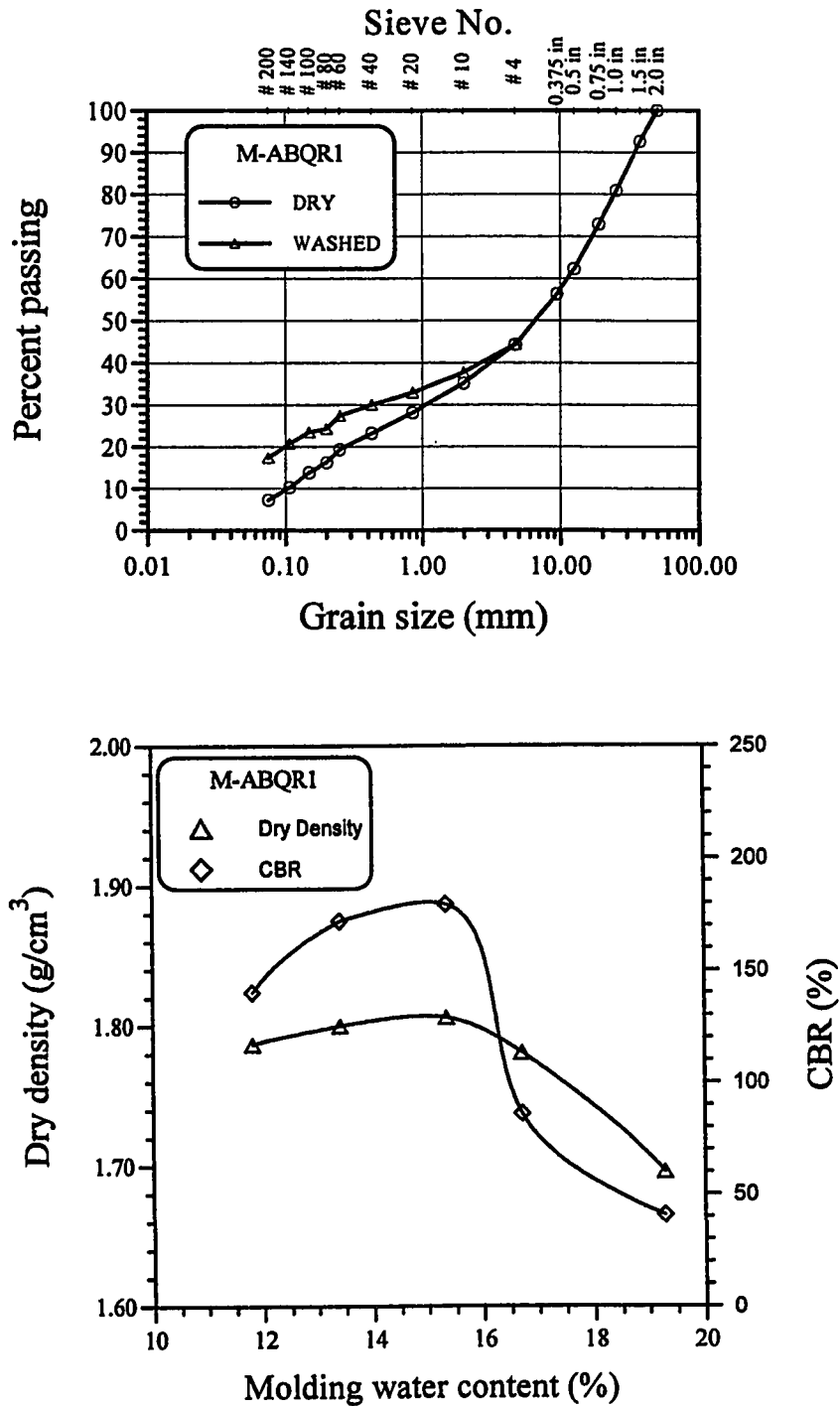


Figure 4.12: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABQR1.

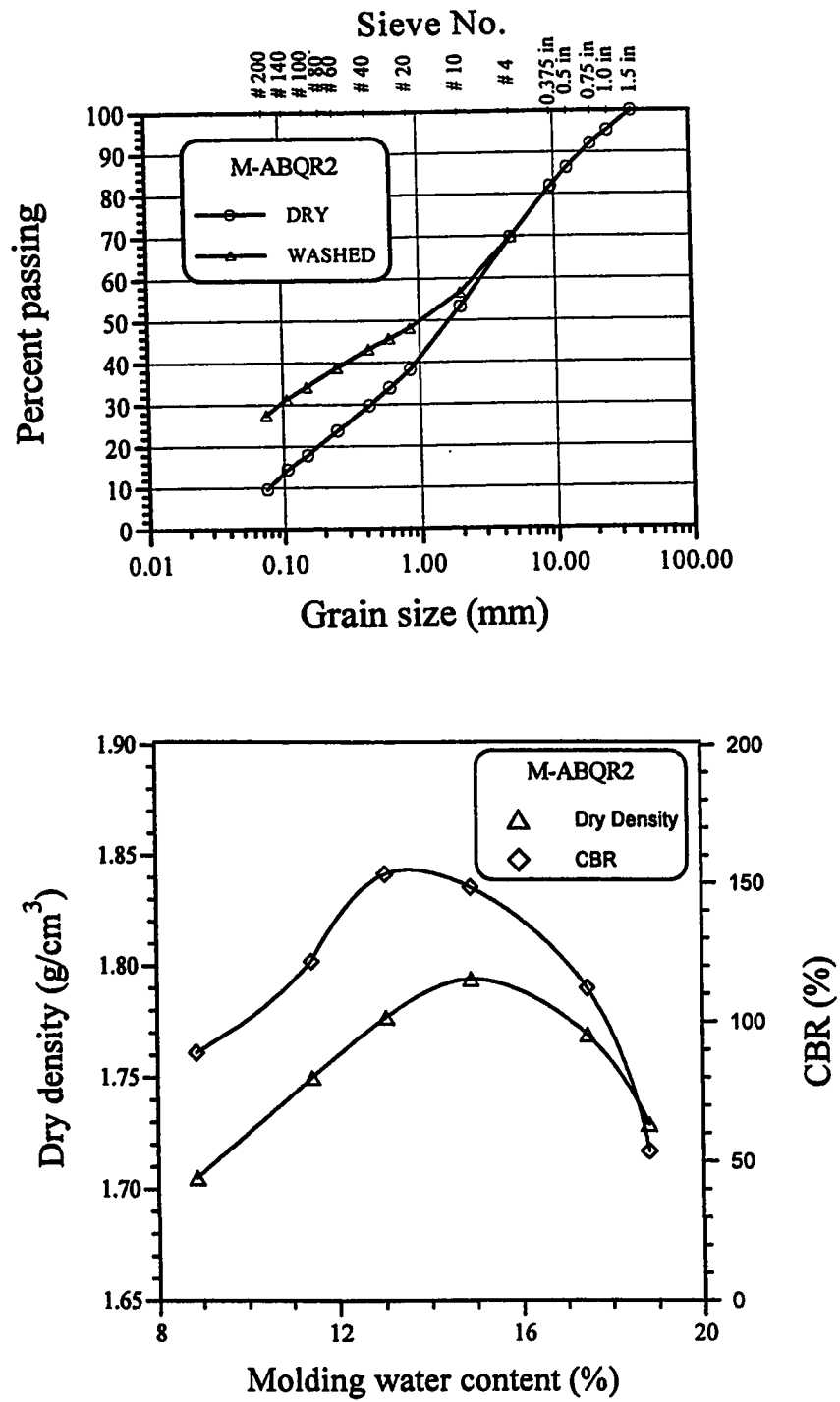


Figure 4.13: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABQR2.

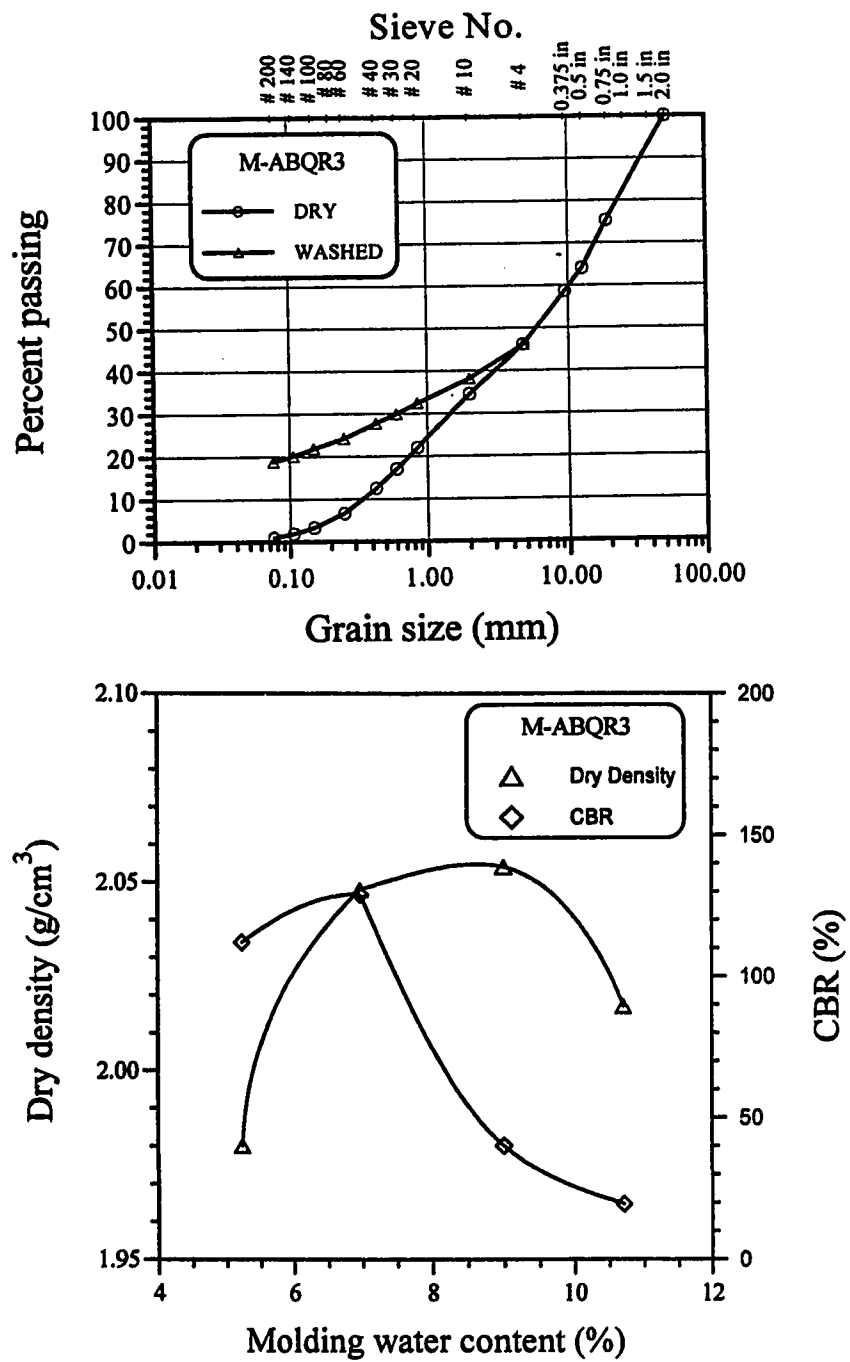


Figure 4.14: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-ABQR3.

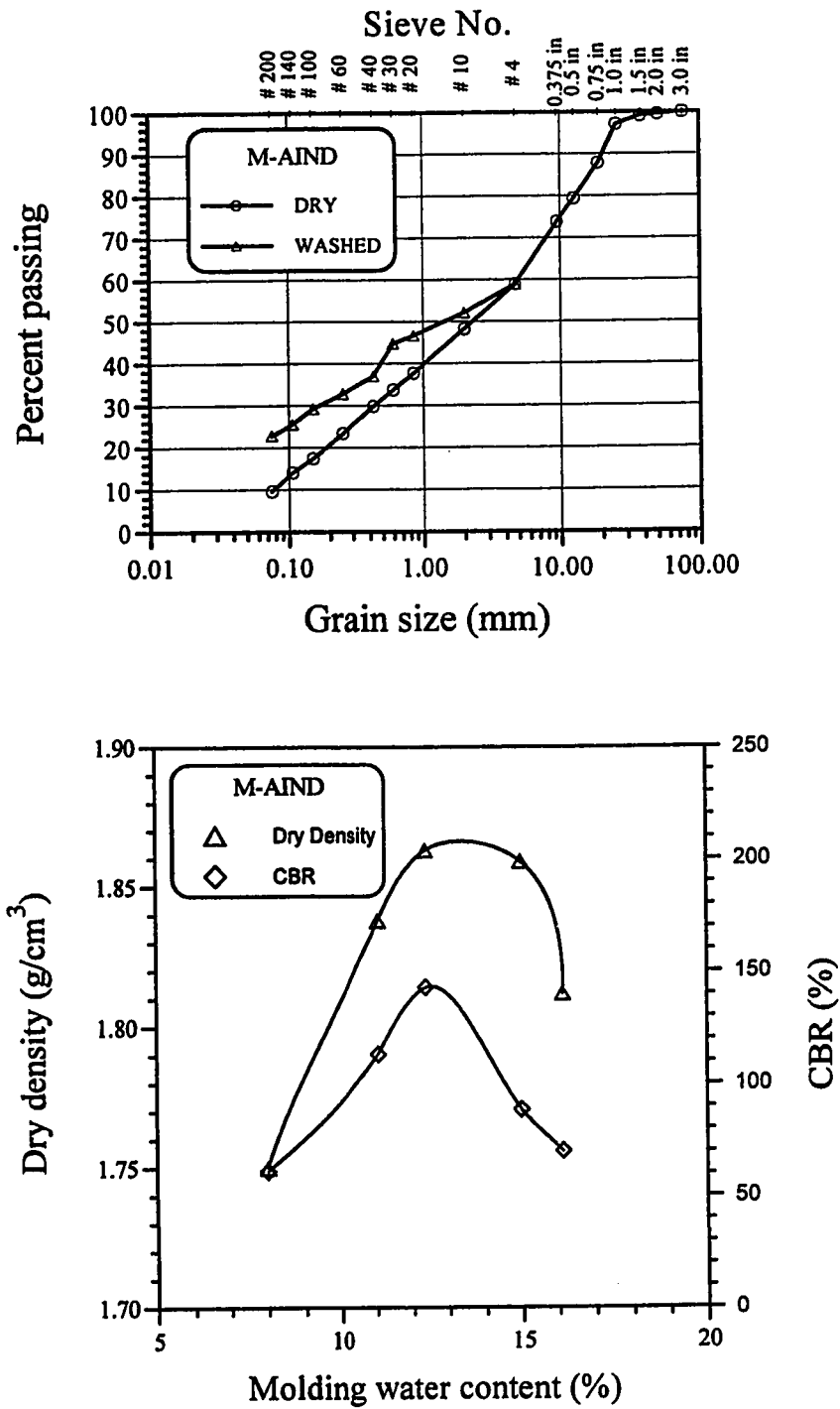


Figure 4.15: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-AIND.

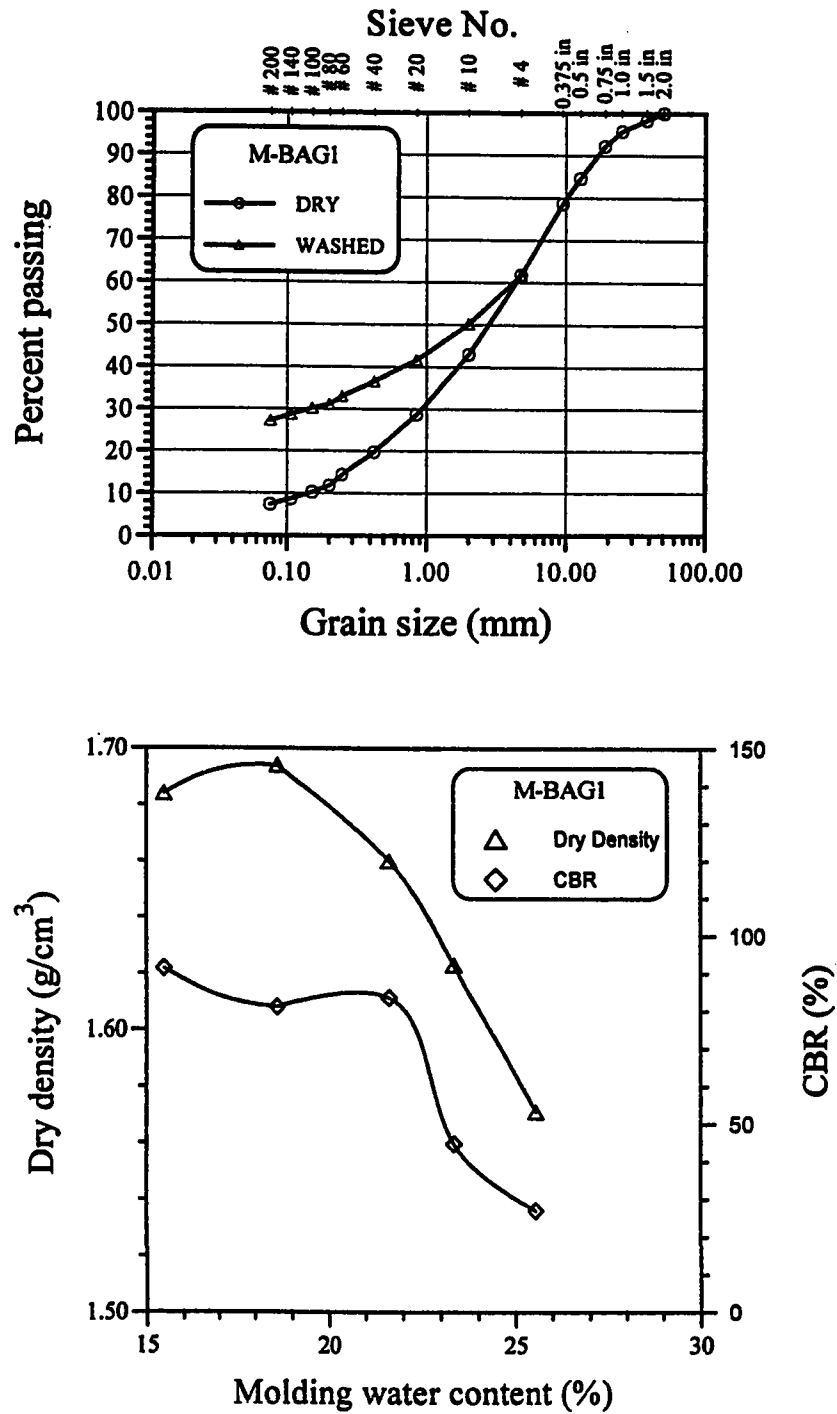


Figure 4.16: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-BAG1.

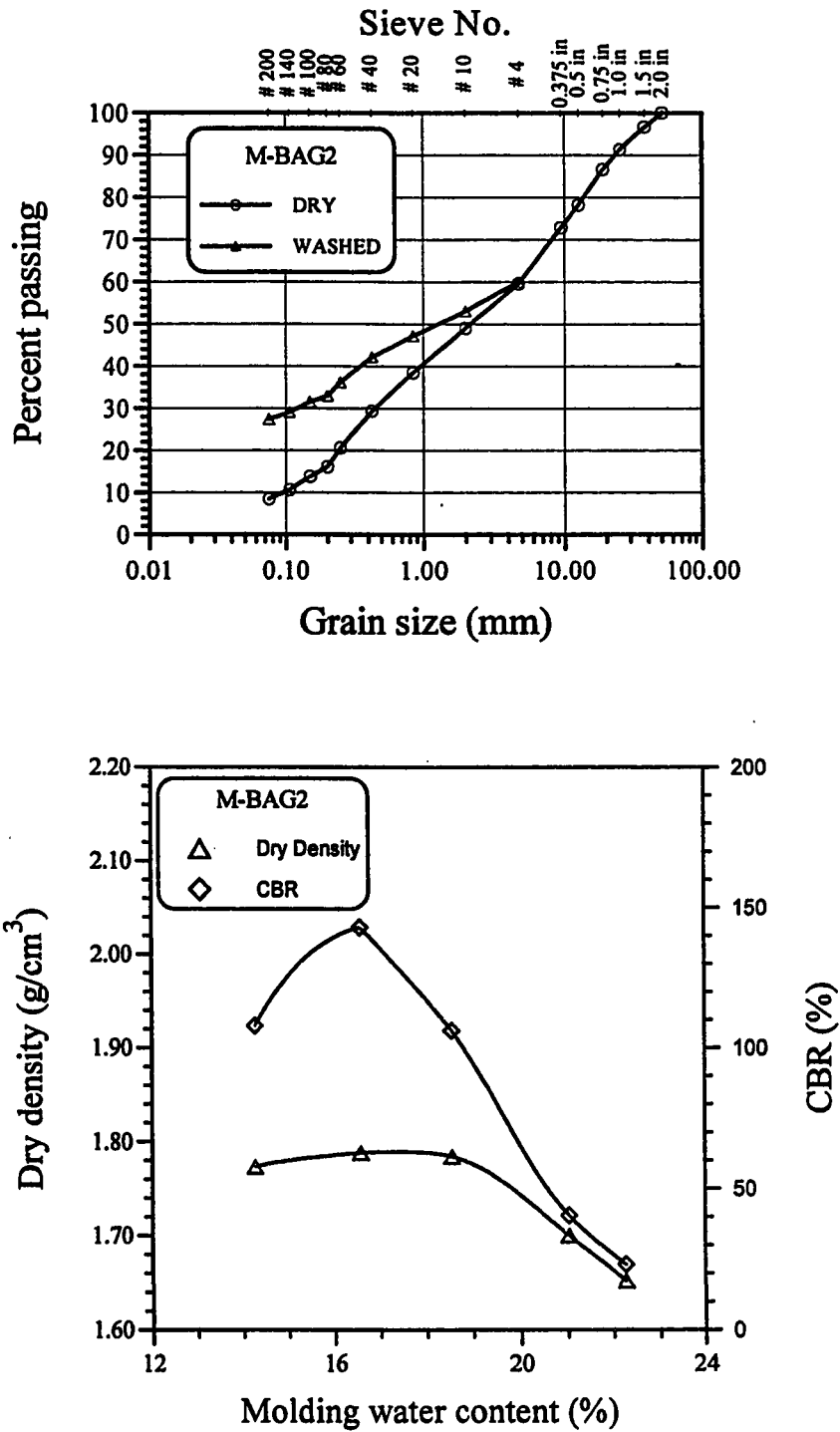


Figure 4.17: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-BAG2.

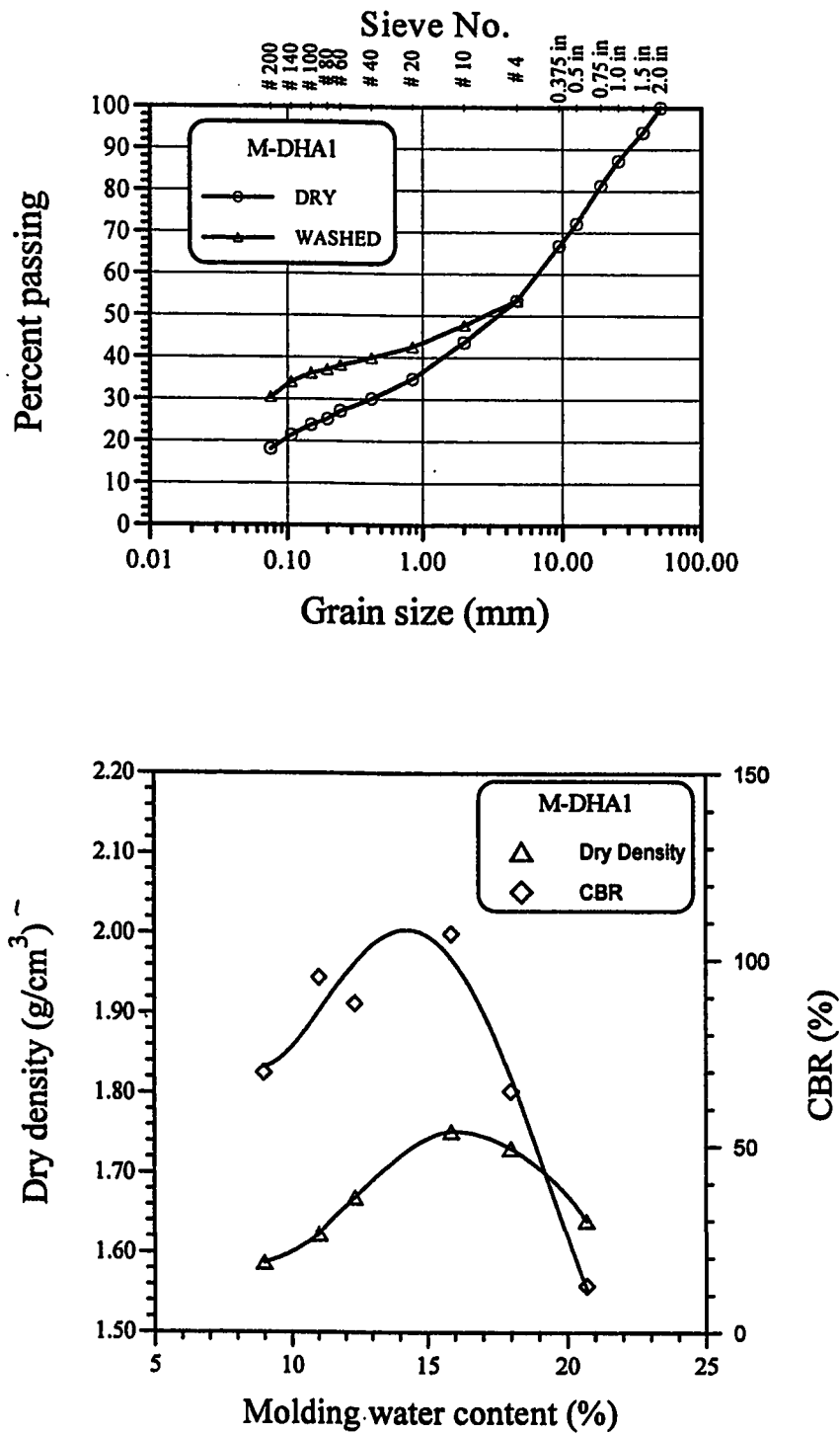


Figure 4.18: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-DHA1.

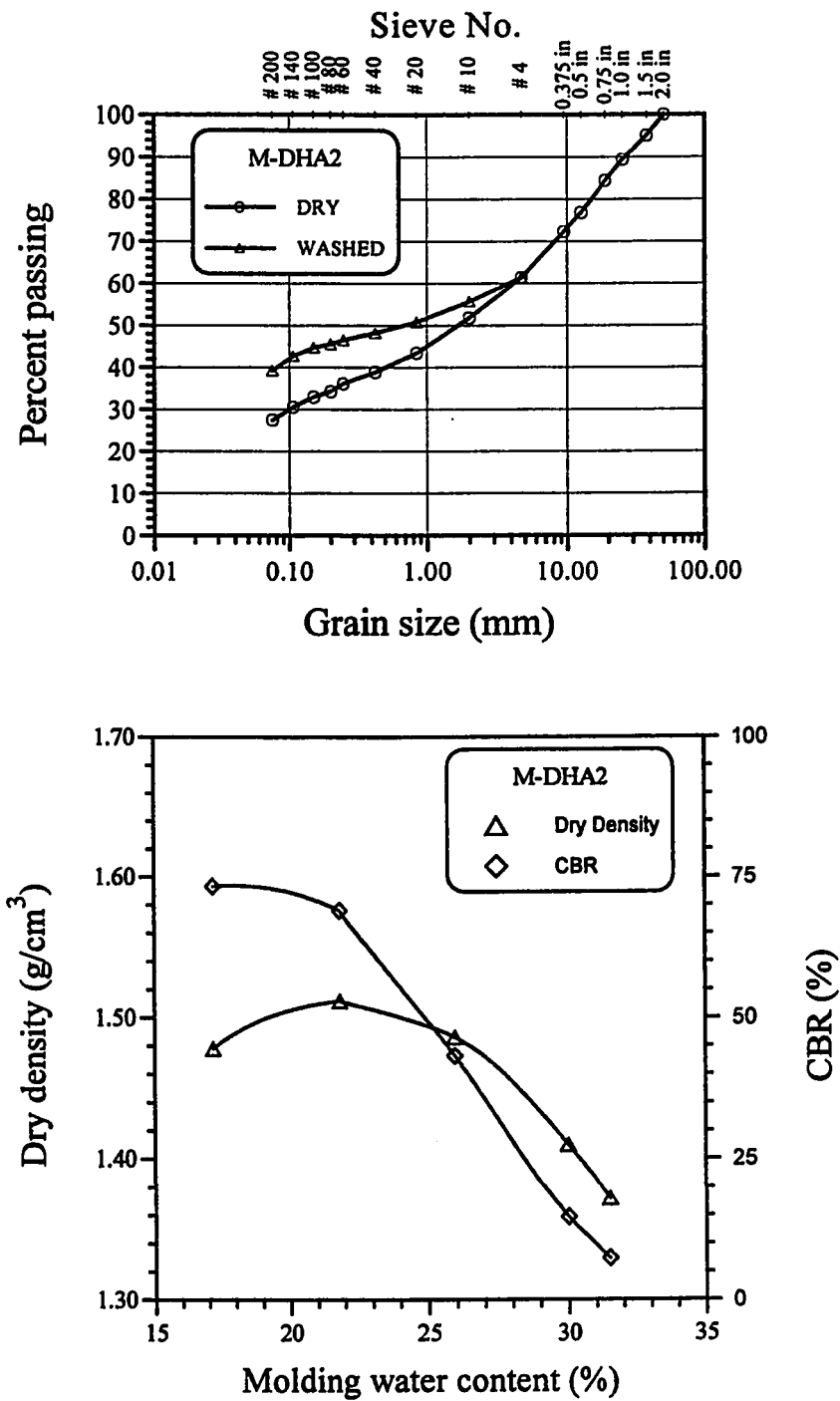


Figure 4.19: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-DHA2.

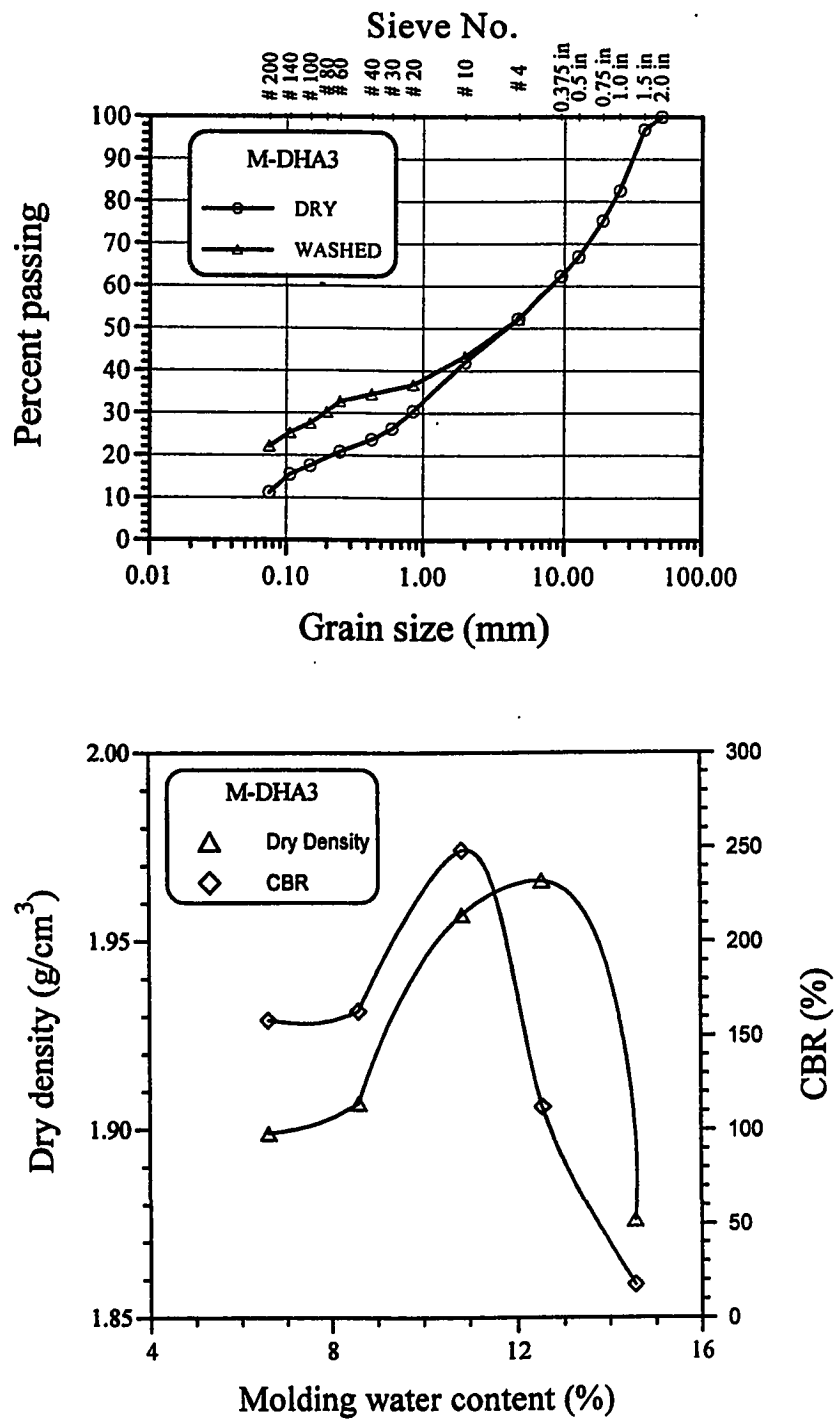


Figure 4.20: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-DHA3.

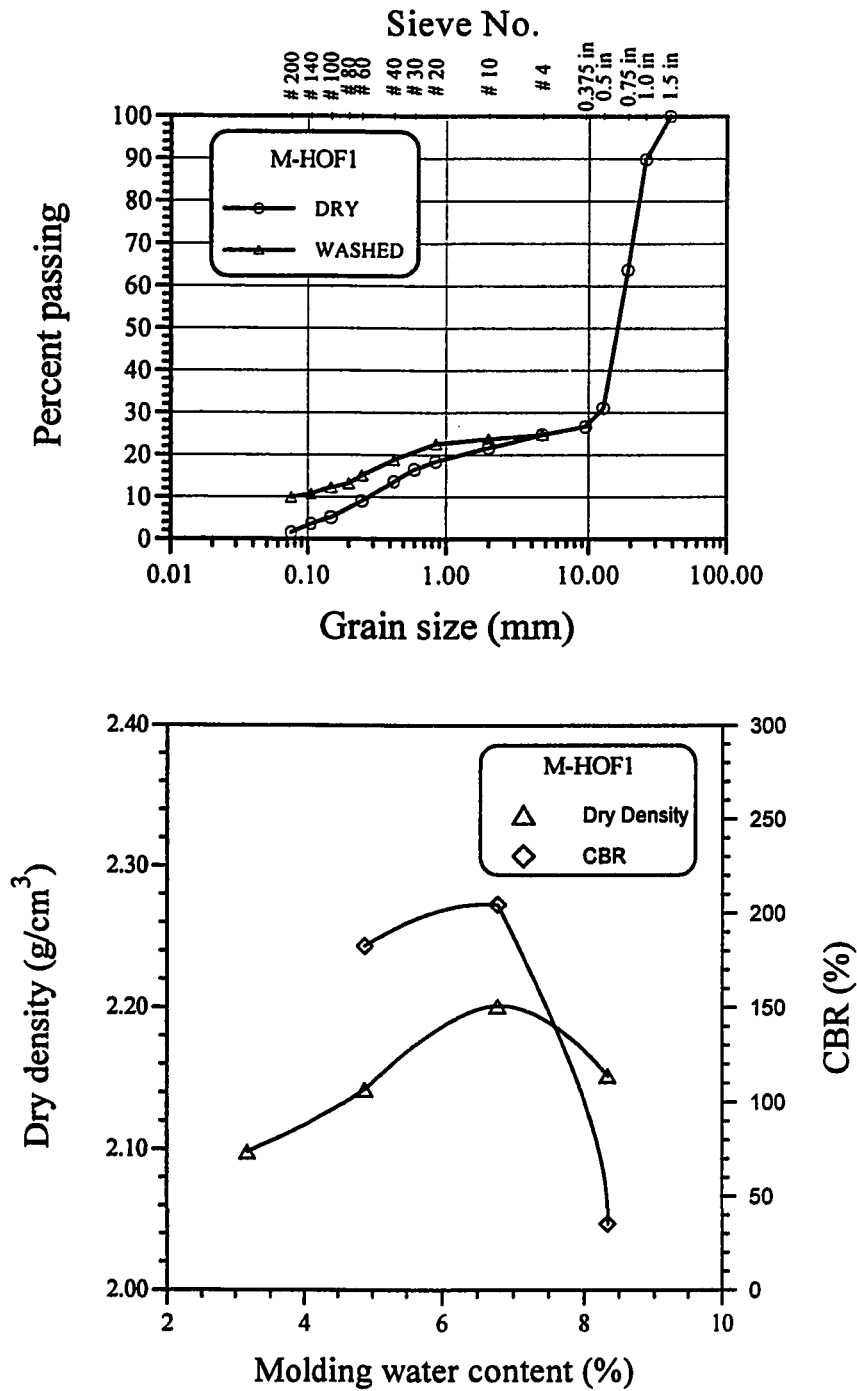


Figure 4.21: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-HOF1.

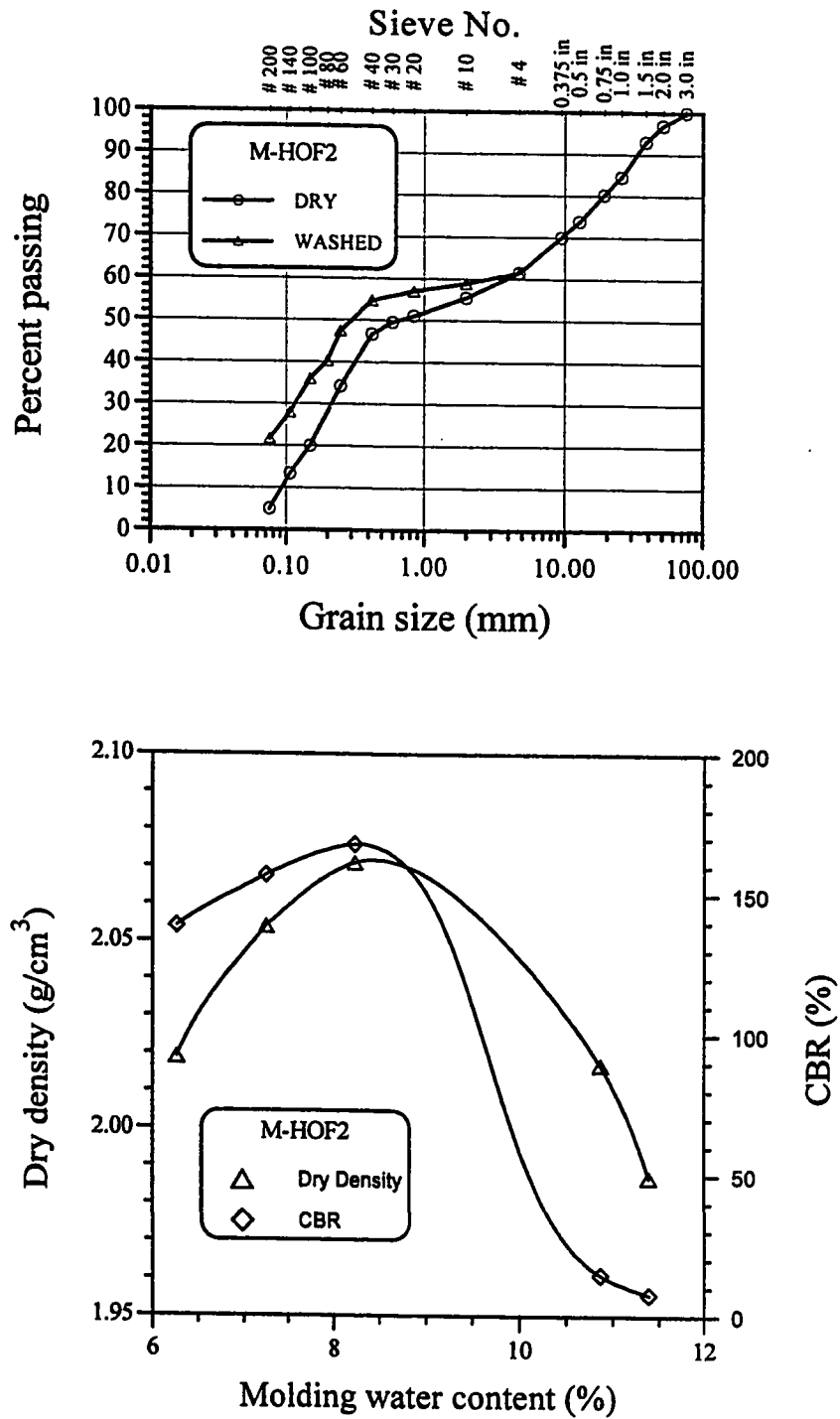


Figure 4.22: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-HOF2.

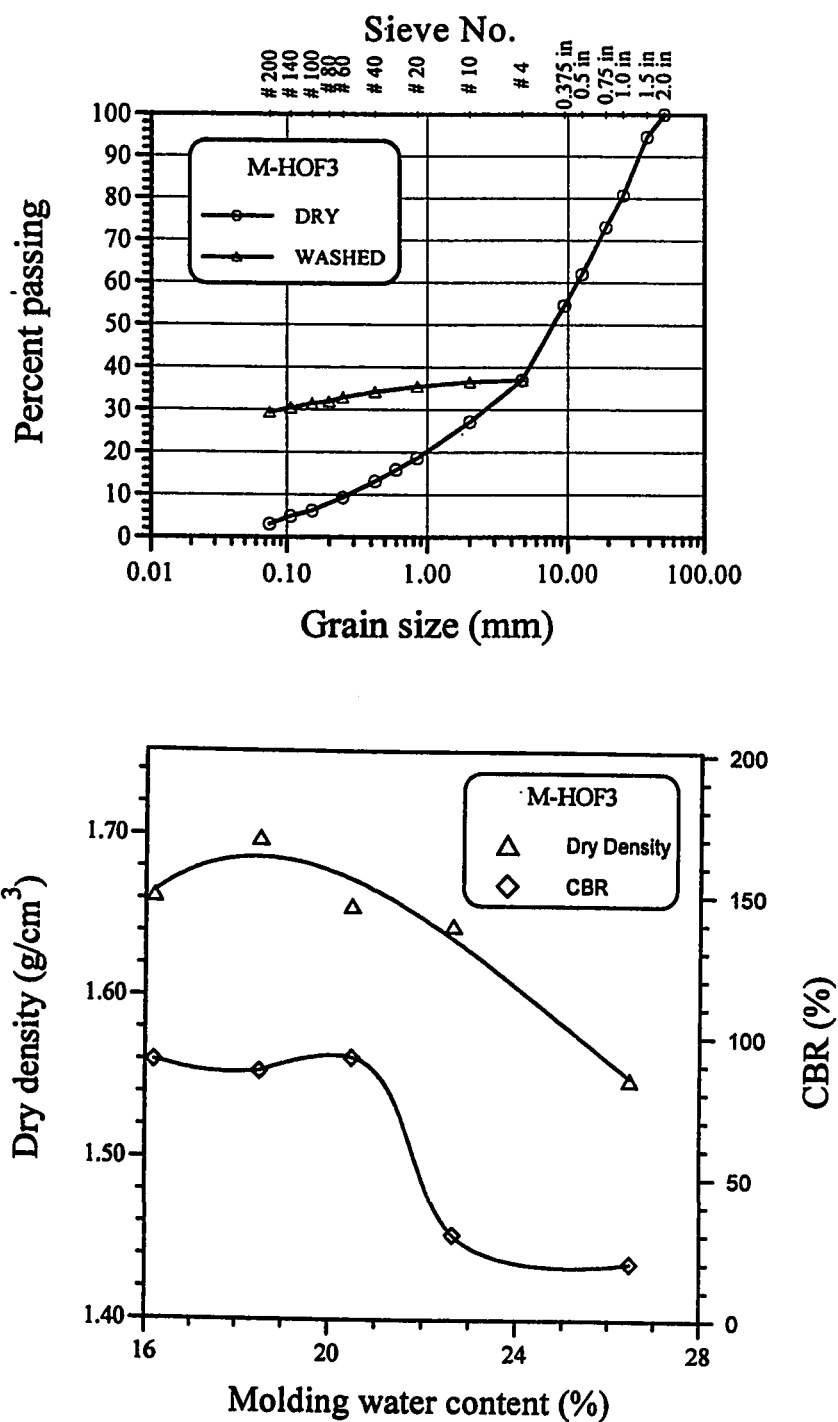


Figure 4.23: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-HOF3.

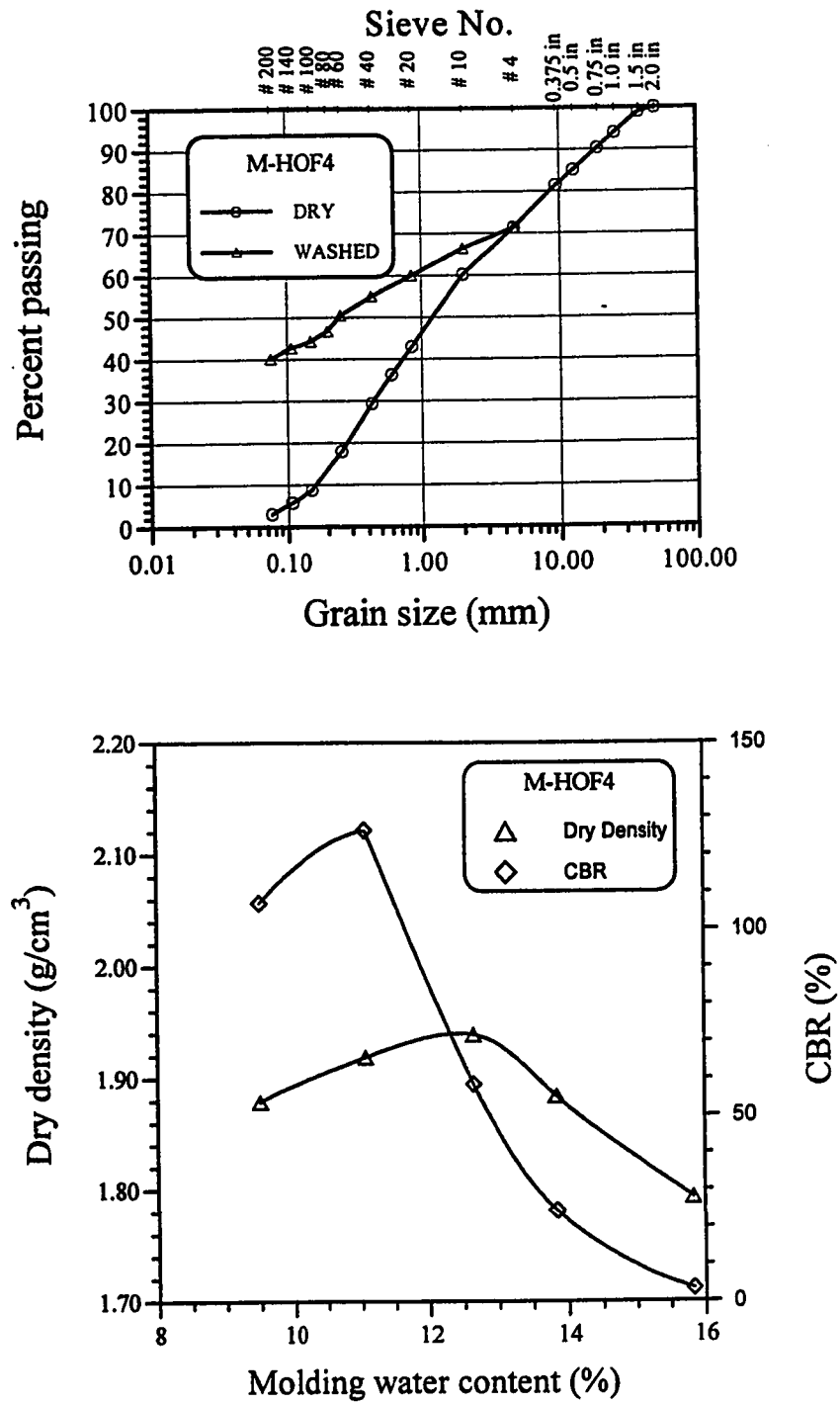


Figure 4.24: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-HOF4.

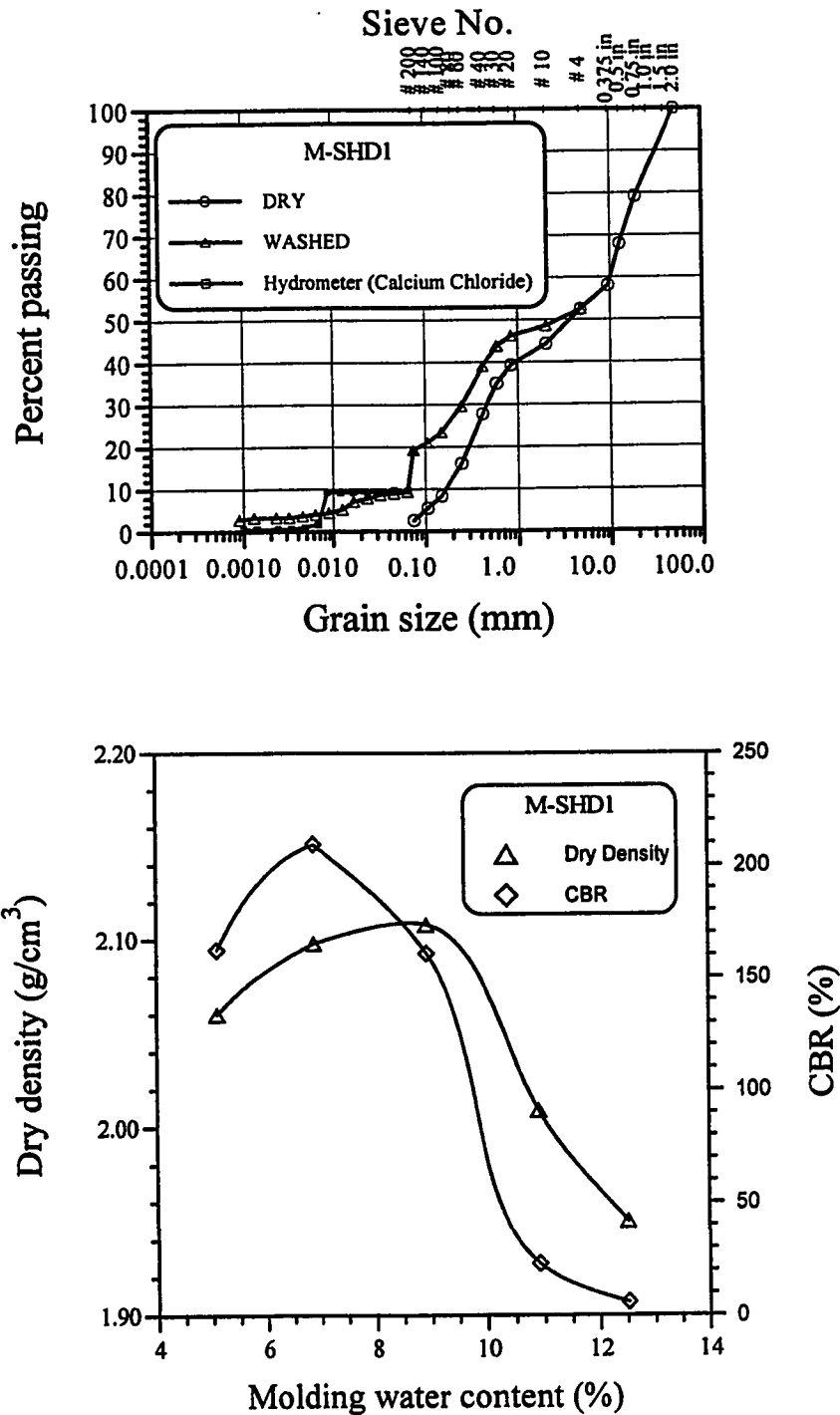


Figure 4.25: Grain-Size Distribution and Moisture-Density-CBR Relationship for M-SHD1.

Modified Proctor Compaction Tests

The engineering properties of compacted soils also depend on the compaction energy and method, and molding moisture content. The moisture-density relationship reflects the behavior of soils during compaction. The soils to be used as fill materials need to be compacted to a certain dry density at a specified moisture content. These two parameters control the structure of the soil which is directly related to the properties of the soil such as strength, compressibility, and permeability. Compacting a soil to the same dry density but at different molding moisture contents produces entirely different fabrics, and consequently the properties become different. For the collected marl samples, the maximum dry density varied from 1.69 to 2.20 g/cm³, and optimum moisture content (OMC) varied from 6.4 to 18.5%. The variability in these parameters reflect the variability of the mineral constituents, grain-size, and their relative percentages in the eastern Saudi marls. Generally, higher optimum moisture contents and lower dry densities are associated with poorly-graded soils, or due to the presence of plastic soil particles.

CBR Tests

In general, soils compacted on the dry side of optimum bear higher strength, than those which are compacted on the wet side of optimum. The higher strength on the dry side owes to the formation of large-sized strong clods (macropeds) which provide frictional resistance, and to the small increase in effective stresses due to suction. At the optimum moisture content, the macropeds are smaller and weaker causing a reduction in the cohesion and thus in the strength. Moreover, the higher density at optimum moisture content cannot counterbalance the less pronounced effects of the macropeds formation. For marl soils, however, maximum CBR val-

ues are obtained either at optimum or very close to the optimum on the dry side. Further away from the optimum on the dry and wet sides of optimum, the strength decreases.

The anomalous behavior of marls on the dry side of optimum may be attributed to the absence of any cohesive material, and therefore the macropeds are friable and relatively weak. At or near the optimum moisture content, denser macropeds give high interlocking of soil aggregate, which is responsible for the strong and stable soil mass. As the moisture content decreases below the optimum, the interlocking decreases due to the reduction in dry density. Since there are no cohesive fines in the soil, strength is derived only through partial interlocking. Similarly on the wet side of optimum, the non-cohesive carbonate fines form loose lumps and the excess water results in a loss of cementation near contacts. The soil mass becomes mud-like lumps with no bearing strength. The gravel-sized aggregate will just float in the loose matrix (lumps) of fines. Aiban (1995) related the strength variation of marl, obtained from Abqaiq area, to the fabric variation at different molding moisture contents. Scanning electron micrographs (SEM) indicated that on the dry side, the macropeds (large-sized group of soil particles known as clusters, cluster in turn is the group of aggregated or flocculated submicroscopic soil particles, termed domains (Holz and Kovacs, 1981)) become bigger in size and macropores (large pores existing in between macropeds) become larger; and small macropeds act as connectors between the larger ones. The weak cementation provided by the connectors, and the presence of large macropores are responsible for the unstable structure for the samples compacted on the dry side of optimum. Increase in moisture content makes the connectors weaker and the macropeds smaller in size.

The strength (CBR) of marl soils varies widely over a narrow range of moisture content. At or near the optimum moisture content on the dry side, the CBR values are well above 100% for all the marl soils, indicating very high strength. In most cases, increasing the moisture content above optimum by about 1% causes complete loss of the soil bearing strength. On the wet side of optimum, the soil forms mud-like lumps and the CBR value drops to almost zero. This behavior was observed with most of the marl samples. However, some marls showed strength and stability even on the wet side of optimum. These marls showed CBR values of greater than or equal to 40% even at a moisture content of 4% above OMC. Based on these findings, the marl samples showing poor behavior on the wet side can be classified as '**marginal marls**', while those showing strength and stability on the wet side can be termed '**satisfactory marls**'. Marl samples collected from Abu Hadriyah and Hofuf areas are 'marginal marls', while those from Abqaiq, Baggah, and Dhahran regions are 'satisfactory marls'.

In the characterization stage, the variation of CBR with the molding moisture content was taken as an indication of the strength behavior for the collected marl samples. Marl soils, classified as A-1-a (AASHTO) or GW (USCS), show an abrupt decrease in strength (CBR) on the wet side of optimum moisture content (OMC). A mere increase of about 1% moisture content above OMC, caused a complete collapse of these soils. On the other hand, marls classified as A-2 showed a much better strength even on the wet side of OMC, upto a moisture content of about OMC plus 4%.

Plastic soils are considered unsuitable in pavement structures, and there is always a maximum plasticity index (PI) that should not be exceeded. It has been observed that the marl soils of eastern Saudi Arabia, which are plastic in nature can be classified as 'satisfactory marls'; while non-plastic marls show a complete loss of strength on the wet side of optimum or upon inundation when compacted on the dry side of optimum, and can be termed 'marginal marls'.

The preliminary results obtained in this research show that the classification systems developed for normal soils in general (USCS and AASHTO), are not strictly applicable to calcareous soils. Classification systems for the calcareous soils, given in Section 2.3, might not be suitable for the marl soils in the Eastern Province. The classification systems provided by Fookes and Higginbottom (1976) and Clark and Walker (1977) are not able to predict the engineering behavior of these soils. Further, the system given by Datta et al. (1982), provides a good way of describing calcareous soils, but they did not provide information where the description can be used to predict the behavior of these soils. Therefore, there is a need for developing a new classification system for locally available marl soils or some modification to the existing soil classification systems. The behavior of calcareous soils seems to be controlled by several factors including the type and content of calcareous and non-calcareous constituents.

The existing classification systems (USCS and AASHTO) have certain presumptions. One of these is that the clay-sized particles are always clayey or plastic in nature, and the other is the plasticity of the soil results from the clay miner-

als only. These assumptions do not apply to calcareous soils. In such soils, the carbonate particles exist as clay-sized particles which are non-plastic in nature. Moreover, the source of plasticity can be the presence of lime or some clay minerals such as attapulgite, which are non-expansive in nature. Although exhibiting high plasticity indices, attapulgite is stable and does not swell (Horta, 1988). So, the plasticity which is an indication of low strength in normal soils may not imply the same thing for calcareous soils.

The new modified classification system should consider the type and nature of the fine-grained calcareous particles. Chemical tests such as carbonate content and lime content should be included, in addition to the index tests which are routinely used to classify soils. In light of this, the classification symbols assigned to a soil can be subscripted using certain additional symbols. These symbols can be 'm' for marginal and 's' for satisfactory. For instance, a calcareous soil classified as GP according to USCS can be named GPm or GPs depending upon the results obtained from chemical or other relevant tests.

4.3 Selection of Marls for Stabilization

One of the main objectives of the characterization stage is to define the engineering behavior of marl soils in general, and to choose some of these marls for further characterization and stabilization studies. The basic considerations in the selection were to select marl types that are not only requiring improvement but also representing the behavior of other marl soil groups in eastern Saudi Arabia. In addition, the selected marls should be available in sufficient quantities for future use in construction.

Abu Hadriyah area is considered one of the main sources of marl in the Eastern Province of Saudi Arabia. Most of the supply to Jubail and Dammam areas come from the many borrow areas in Abu Hadriyah. Soil from some of the borrow areas in Abu Hadriyah are used by the Dammam municipality for construction and maintenance of local roads and highways. Although the soil from Abu Hadriyah is being widely used, its behavior in base and sub-base layers is poor. When this marl is used, complete failure of roads is commonly observed. A complete loss of bearing capacity occurred on the wet side of optimum, or upon flooding the samples prepared on the dry side of optimum. These might be the probable causes of failure in the field, besides the heavy loading. Since it is difficult to accurately control the molding moisture content in the field, the compaction of marl soil slightly on the wet side of optimum is inevitable. This practice cannot be considered normal for such soils. Moreover, inundation of the compacted layers at a later stage by ground or surface water is also responsible for the strength loss. Therefore, these soils should be subjected to certain improvement to deprive them from their moisture sensitivity. Considering the above factors, it was decided to choose one marl type from Abu Hadriyah area (M-ABH9) for detailed characterization and stabilization.

The marl obtained from Shedgum area (M-SHD1) is being used by Saudi ARAMCO for their construction. The soil consists of natural aggregates of different sizes with an acceptable gradation. It is a weathering product of duricrust existing in the Shedgum area. Since the mode of formation of this soil was different from the rest of the collected marls, it was considered a suitable candidate

for detailed study. Its water sensitivity is almost of the same order as that of Abu Hadriyah materials. Therefore, it was selected as a second marl for detailed investigation. This marl was reconstituted to the gradation specified by Dammam Municipality.

4.4 Detailed Characterization

The selected marls were subjected to detailed characterization before the stabilization stage. Chemical and mineralogical analysis as well as soaked CBR and oedometer tests were conducted on the selected marls, as part of the detailed characterization.

4.4.1 Chemical Analysis

Chemical tests were performed to determine the calcium carbonate and calcium magnesium carbonate contents of the selected marls. Material passing ASTM #40 and #100 sieves, obtained using dry sieving, were subjected to EDTA analysis. The results are presented in Table 4.4, and the following comments are observed:

1. Abu Hadriyah marl has both calcite and dolomite, with calcite as the primary mineral and dolomite as the trace mineral.
2. Shedgum marl contains a high percentage of calcite, while dolomite is not present.
3. Soil passing sieve ASTM #100 gave higher percentages of calcite than that passing #40 sieve. This shows that the calcite exists either naturally as fine particles, or it was weathered easily to finer sizes.

Table 4.4: Chemical Analysis of Selected Marl Soils

Marl	Fines	Ca (%)	Mg (%)	CaCO ₃	CaMg(CO ₃) ₂
Abu-Hadriyah (M-ABH9)	- * #40	42.8	3.8	39.0	3.8
	- #100	56.3	0.0	56.3	0.0
Shedgum (M-SHD1)	- #40	37.5	0.0	37.5	0.0
	- #100	62.6	0.0	62.6	0.0

* - implies passing sieve #

4.4.2 Mineralogical Analysis

Identification of the minerals present in the soil is useful in correlating the engineering behavior with the existing minerals. Each mineral has its own response to the physical and environmental conditions. Mineralogical analysis of the selected marls was carried out using X-Ray diffraction (XRD) technique.

Table 4.1 shows the different minerals present in the selected marls. The XRD micrographs for both marls are shown in Figs. 4.26 and 4.27. Analysis of Abu Hadriyah marl results indicates a very high percentage of calcite, while dolomite is present in traces. Shedgum marl has a high percentage of calcite and quartz, while dolomite is absent. It should be clear that the XRD results are greatly affected by the heterogeneity of the soil and needed to be correlated with the chemical analysis results. Based on the mineralogy of selected marls, these soils cannot be labeled as marl due to the absence of clay. However, it is better to call them 'calcitic soils' since calcite is the main mineral present in these soils. The source of quartz in these calcitic soils may be chert, flint or similar materials. Traces of sepiolite clay mineral in Abu Hadriyah marl and Palygorskite in Shedgum marl indicate the presence of clayey minerals as traces in these carbonate soils. These clayey particles were either deposited along with the carbonate particles or transformed later from the existing minerals under certain environmental (temperature, pressure, and nature of pore fluid) conditions.

4.4.3 Soaked CBR

Soaked CBR was performed to investigate the effect of inundation of marl samples. It is known that the strength of soils decreases upon inundation. In the field, the

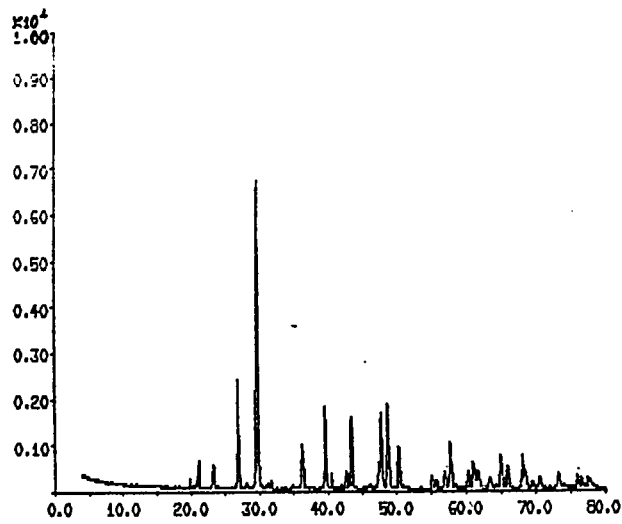
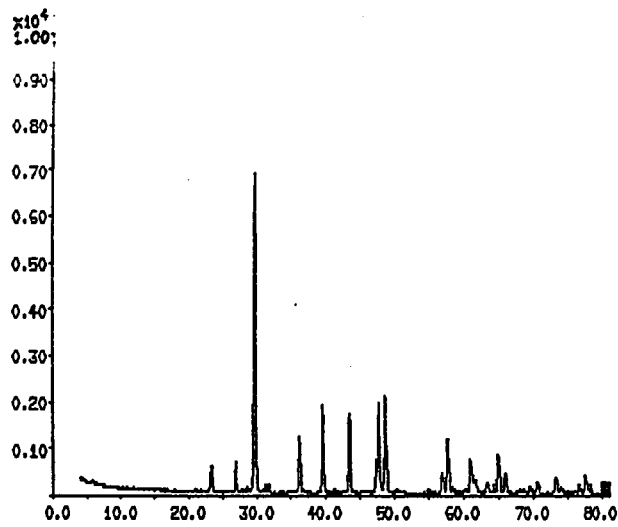
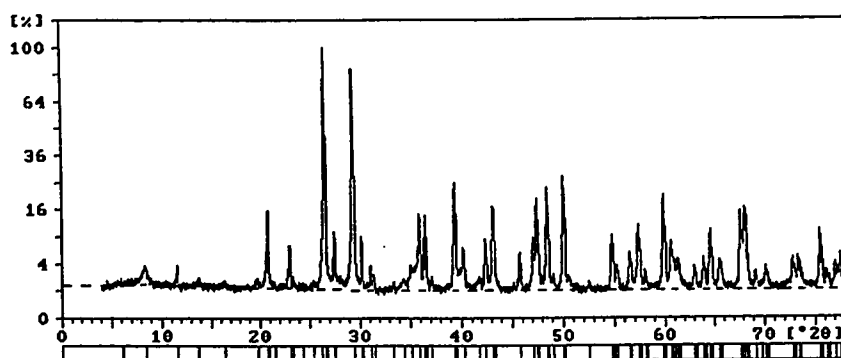
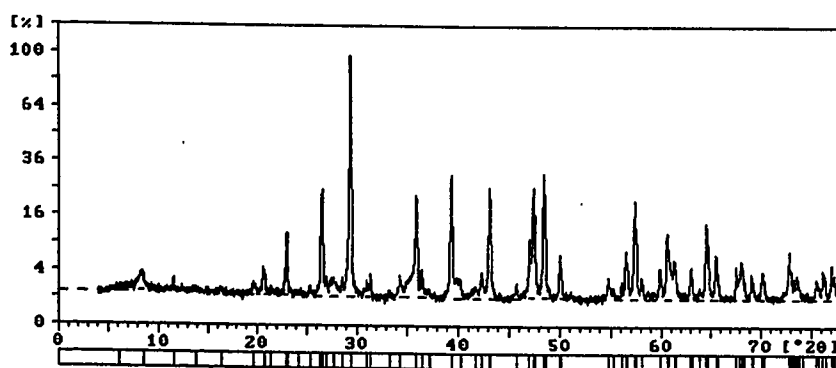
**a****b**

Figure 4.26: XRD Micrographs for M-ABH9 for Soil Fraction Passing a) ASTM Sieve No. 40 b) ASTM Sieve No. 100.



a



b

Figure 4.27: XRD Micrographs for M-SHD1 for Soil Fraction Passing a) ASTM Sieve No. 40 b) ASTM Sieve No. 100.

sources of inundation include surface and ground waters, as well as leakage from adjacent septic tanks, water tanks, sewers, and underground pipes. Soaked CBR values are an indicative of both the strength and the strength loss upon inundation. Soaked CBR tests were performed on M-ABH9 and M-SHD1 marls, and the results were compared with those of the as molded ones.

The variation of soaked and unsoaked CBR values with the molding moisture content, for M-ABH9 and M-SHD1, is shown in Figs. 4.28 and 4.29, respectively. These figures show that the maximum reduction in CBR due to soaking is on the dry side of optimum, and it decreases as the molding moisture content increases above the optimum value. On the dry side of optimum, the reduction in strength is due to the existence of open fabric and to the lower dry density. As the penetration of water on the dry side of optimum is more than that for samples compacted at the optimum and on the wet side of optimum, weakening of the macropeds occurs and consequently the strength decreases (Aiban, 1995). Moreover, saturation is also responsible for the pore pressure built up during stress application. In addition, the water destroys the cementation formed by the precipitation of carbonates and other salts if any. On the wet side of optimum, the CBR values are not affected much by soaking. The permeability of soil samples is very low if compacted on the wet side of optimum. At the optimum, high density ensures low permeability, preventing much water to enter the sample to weaken it and the high density ensures higher strength.

Although the samples compacted on the dry side of the optimum show quite high unsoaked CBR values, they lose much of their strength upon soaking. Soaked

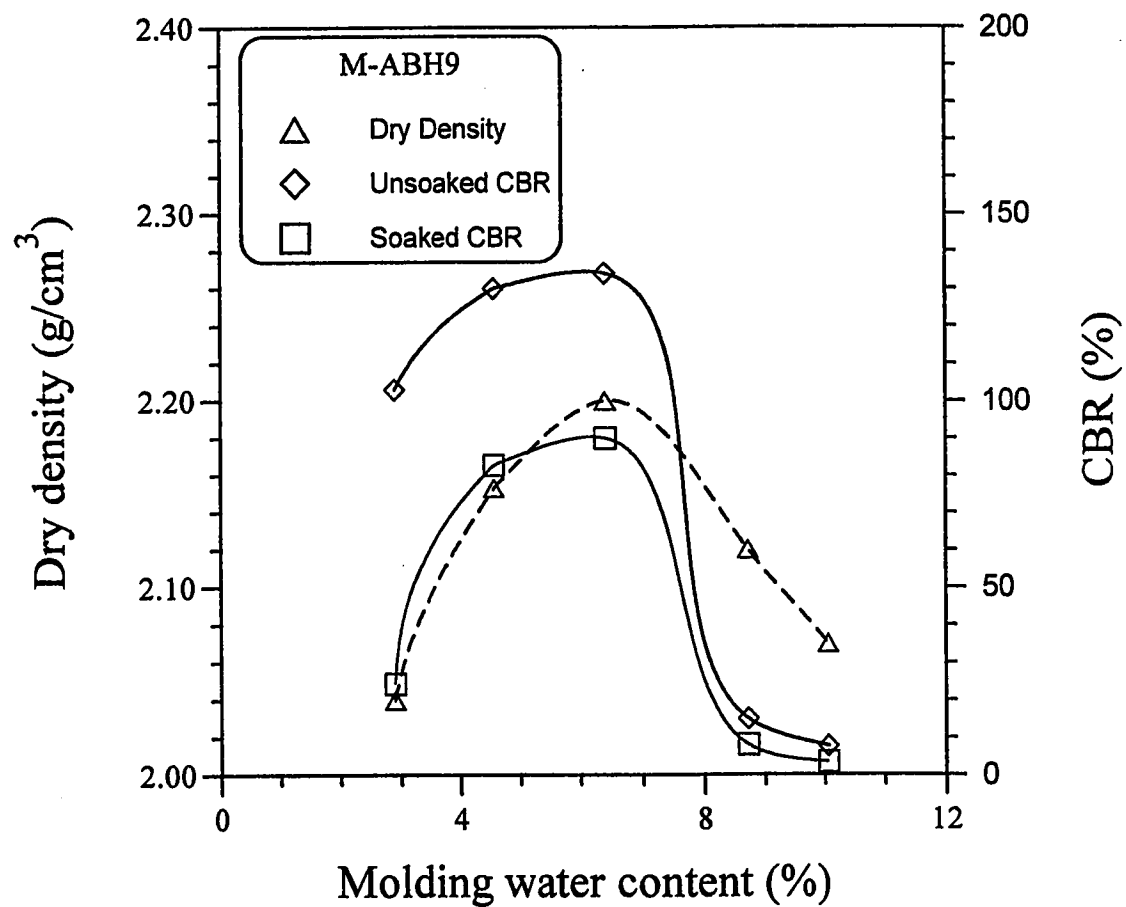


Figure 4.28: Variation of Soaked and Unsoaked CBR with Molding Moisture Content for M-ABH9.

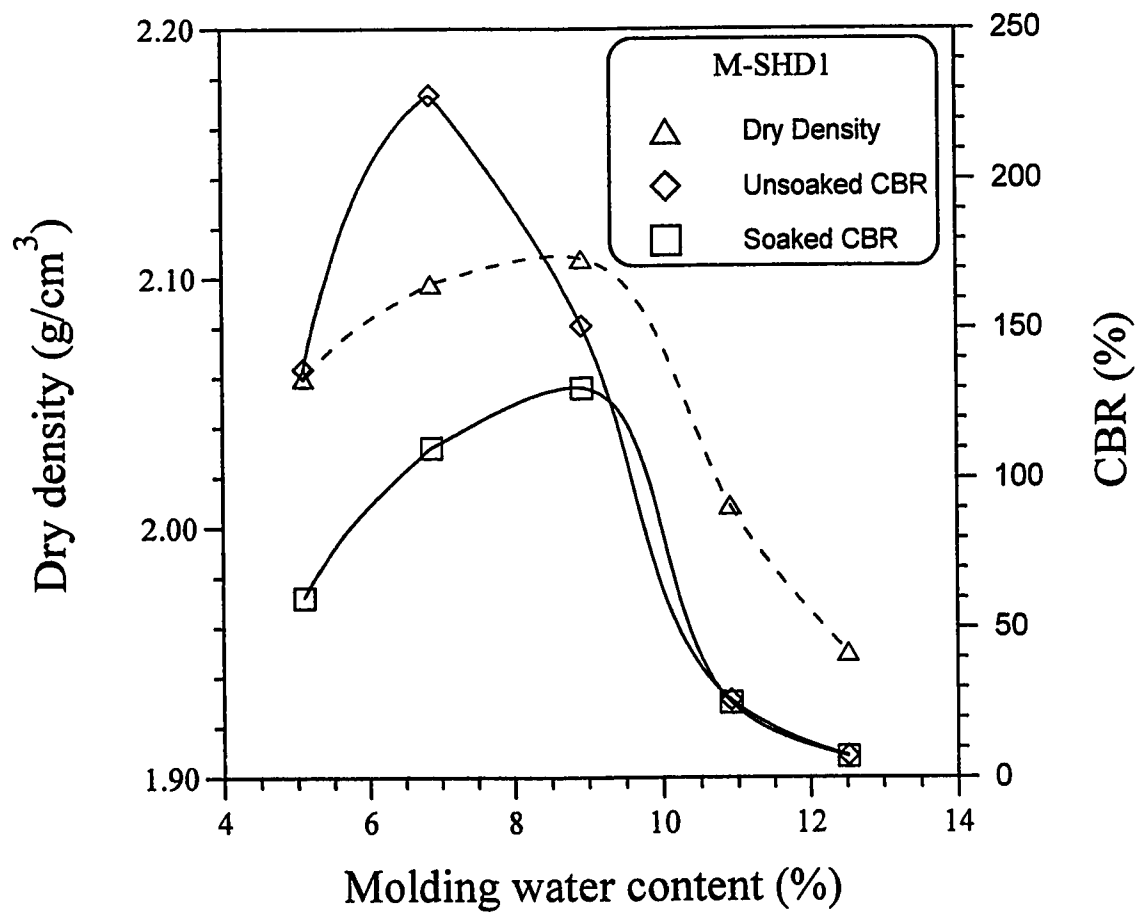


Figure 4.29: Variation of Soaked and Unsoaked CBR with Molding Moisture Content for M-SHD1.

as well as unsoaked behavior of the selected marls has an important design and construction significance. Compaction of marl at the optimum moisture content should always be desired, because a little deviation from that will result in great strength loss. The results of the present investigation consistently caution the extreme sensitivity of the marl soils to the molding moisture content as well as that at testing.

4.4.4 Clegg Impact Value

In the detailed investigation, the samples prepared for CBR testing (i.e. both unstabilized and stabilized) were also tested using the Clegg impact hammer. Clegg hammer was used on the opposite side of samples tested for CBR. The Clegg Impact Values (CIV) were plotted against the corresponding CBR as shown in Fig. 4.30. This plot can be helpful in predicting the CBR values in the field using Clegg hammer which is relatively quick and easy. The best correlation developed between CBR and CIV can be expressed in the following exponential form:

$$CBR = 11.98 * e^{0.039 * CIV} \quad (4.1)$$

The data and hence the relation between the variables, are considered reliable if R^2 is greater than 0.75. The relation (4.1) can be considered reliable, as the value of standard deviation (R^2) is 0.88.

4.4.5 Oedometer Testing

Oedometer tests were conducted on the materials passing ASTM #4 sieve from the selected marls. Test results are shown in Figs. 4.31 and 4.32. The corresponding compression indices (C_c) and rebound or swelling indices (C_r or C_s) are given in Table 4.5. Maximum compression occurred on samples compacted on the wet side

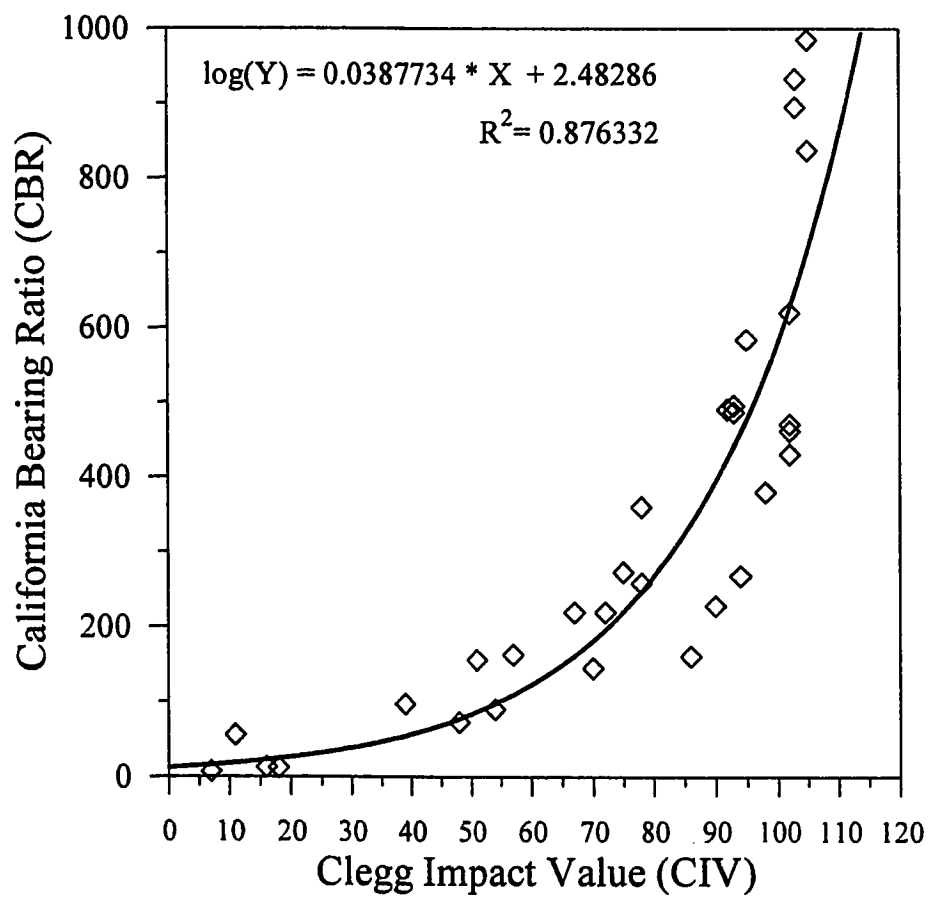


Figure 4.30: Correlation Between CBR and CIV for Stabilized and Unstabilized Selected Marls.

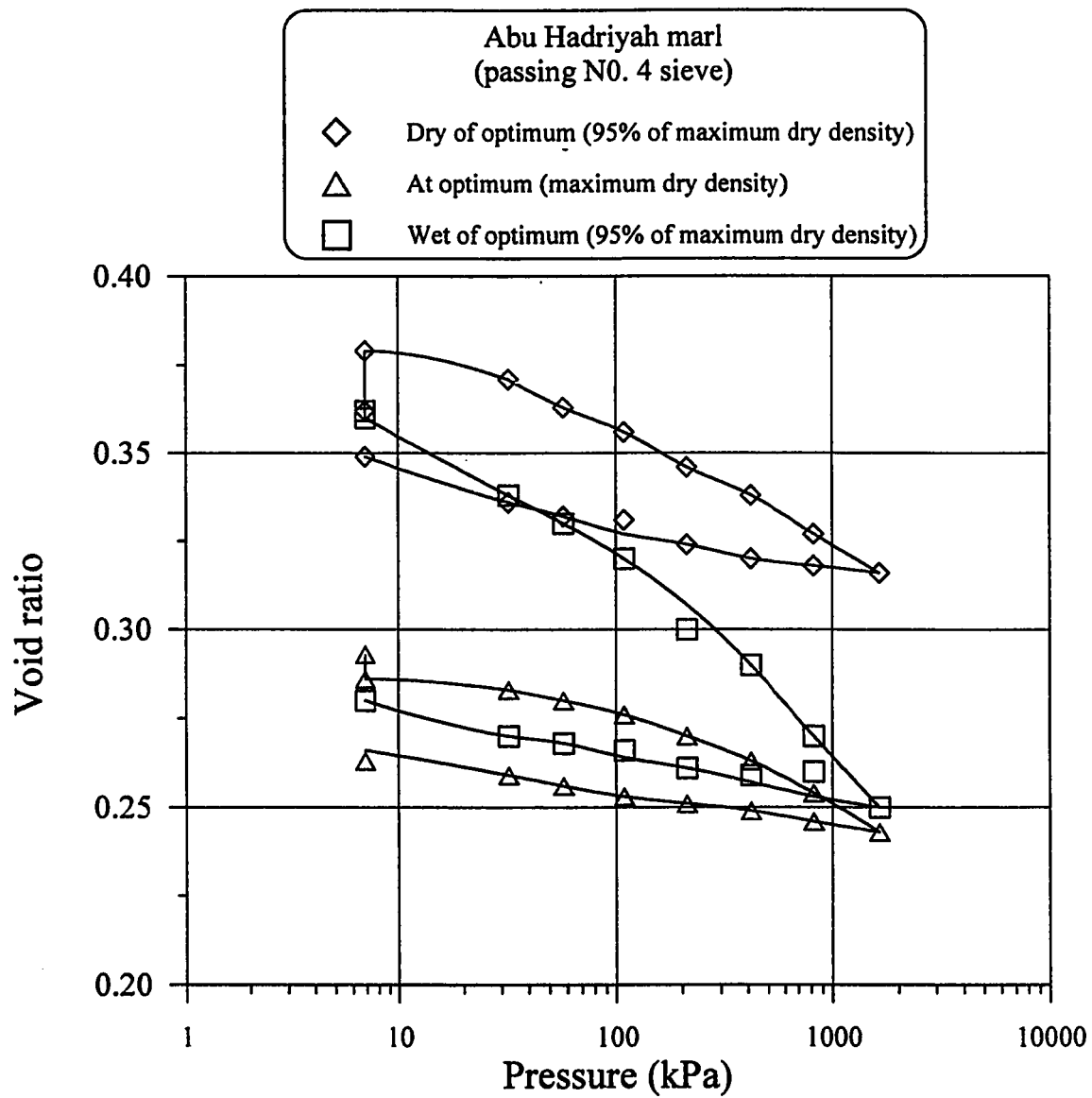


Figure 4.31: Oedometer Testing for M-ABH9.

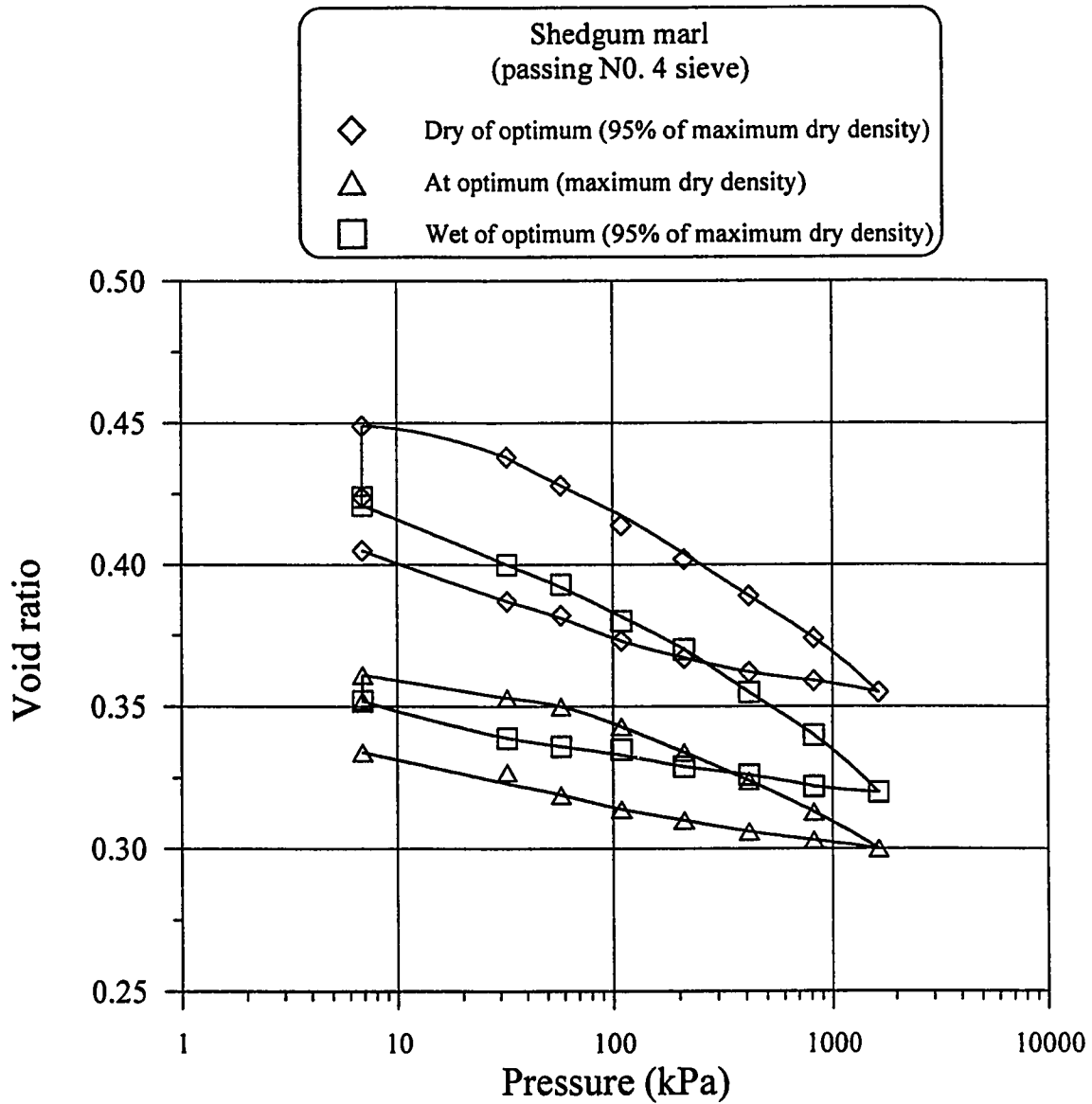


Figure 4.32: Oedometer Testing for M-SHD1.

Table 4.5: Compression and Rebound Indices for M-ABH9 and M-SHD1

Marl	Compacted at	Compression Index (C_c)	Rebound Index (C_r)
M-ABH9	Dry of Optimum*	0.037	0.013
	Optimum	0.033	0.010
	Wet of Optimum*	0.064	0.012
M-SHD1	Dry of Optimum*	0.055	0.021
	Optimum	0.036	0.010
	Wet of Optimum*	0.056	0.016

* moisture content corresponding to 95% of the maximum dry density

of optimum for both marls. This is mainly due to the very weak structure of the samples compacted on the wet side of optimum. The soils compacted on the dry side of the optimum showed an increase in volume under a seating pressure of 6.9 kPa, due to the affinity of fines to water. The increase in volume was 1.35% and 1.75% for Abu Hadriyah and Shedgum marls, respectively. This very low swelling is due to the presence of certain trace minerals such as clay, gypsum, and anhydrite in these marls.

Moreover, it can be seen from the plots that no well-defined straight portion of the compression and swelling curves is obtained. In addition, the C_r/C_c ratios are high for marl when compared to other (non-carbonate) soils.

4.5 Stabilization of Selected Marls

In practice, soil deficient in certain engineering aspects can be improved by employing certain stabilization procedures. Selection of a specific improvement method depends on many factors including: type of soil, nature of the problem and the project, techniques available, economical aspects, and environmental and loading conditions. These factors should always be considered when any of the stabilization techniques are to be used.

4.5.1 Selection of Stabilization Method

Results obtained from the characterization tests gave a good indication about the behavior of marl soils. The main problem associated with these soils is their acute sensitivity to water. The marl soils are very sensitive to the molding moisture and the moisture content at testing. A complete loss of bearing capacity occurs upon inundation.

Several measures can be taken to minimize this phenomenon. Water should not be allowed to come into contact with these soils, or the soil should be made water resistant using certain treatment methods. Isolation of the soil may not be feasible since the source of moisture in the field is usually from both the ground and the surface waters. Drainage is one of the procedures to keep water away from the marl layers. Lowering the ground water table can be achieved by dewatering techniques which are costly and require a continuous monitoring and maintenance.

The drainage of surface waters is always desirable for the strength and stability of pavement layers. This will be possible by collecting and disposing off the surface water using surface catchment drains. If this system is to work effectively for the entire life of pavements, large sums of initial, maintenance and operation costs will be required. This may not solve the rising water table in many Saudi cities, and therefore isolation of the soil from the ground and the surface water may not be practical.

On the other hand, marls can be subjected to mechanical or chemical treatment methods in order to make them water resistant. These methods are not only economical compared to the above alternatives, but also effective in reducing the maintenance costs to almost nil. These methods are given in the following subsections, where usefulness of each technique can be appreciated.

Mechanical Stabilization Using Sand

In this technique, the soil passing ASTM Sieve #30 was replaced by an equivalent quantity of quartz sand, and termed 'modified marl'. CBR testing was used to evaluate the behavior of the original and modified marls. Abu Hadriyah marl (M-ABH9) was chosen to study the effect of this replacement. To eliminate the effect of any change in gradation, the gradation of marl passing ASTM Sieve #4, was compared with that of sand, and soil passing ASTM Sieve #30 was replaced by an equivalent amount of quartz sand on a weight basis. The resulting marl soil was named as "modified Abu Hadriyah marl" and labeled M-ABH9a. The gradation of the sand, M-ABH9, and M-ABH9a are shown in Fig. 4.33. The sand was obtained from Baggah area in the Eastern Province. The modified soil was subjected to CBR and the test results indicate that the CBR values of the original soil and the modified soil (M-ABH9a) were almost identical. For both the original marl and the modified marl, the moisture-density and the moisture-CBR relationships are shown in Figs. 4.34 and 4.35, respectively. There was an increase in maximum dry density and a decrease in the OMC of the modified soil compared to the original one. The increase in the dry density resulted from the replacement of lighter carbonate particles with heavier quartz sand particles, while the decrease in OMC is due to the lesser water required to lubricate the sand-sized quartz particles.

Chemical Stabilization

The selected marl soils were to be stabilized using either portland cement or hydrated lime. A primary step in the selection of chemical stabilizer was to investigate the relative improvement produced by the two candidates. Unsoaked CBR

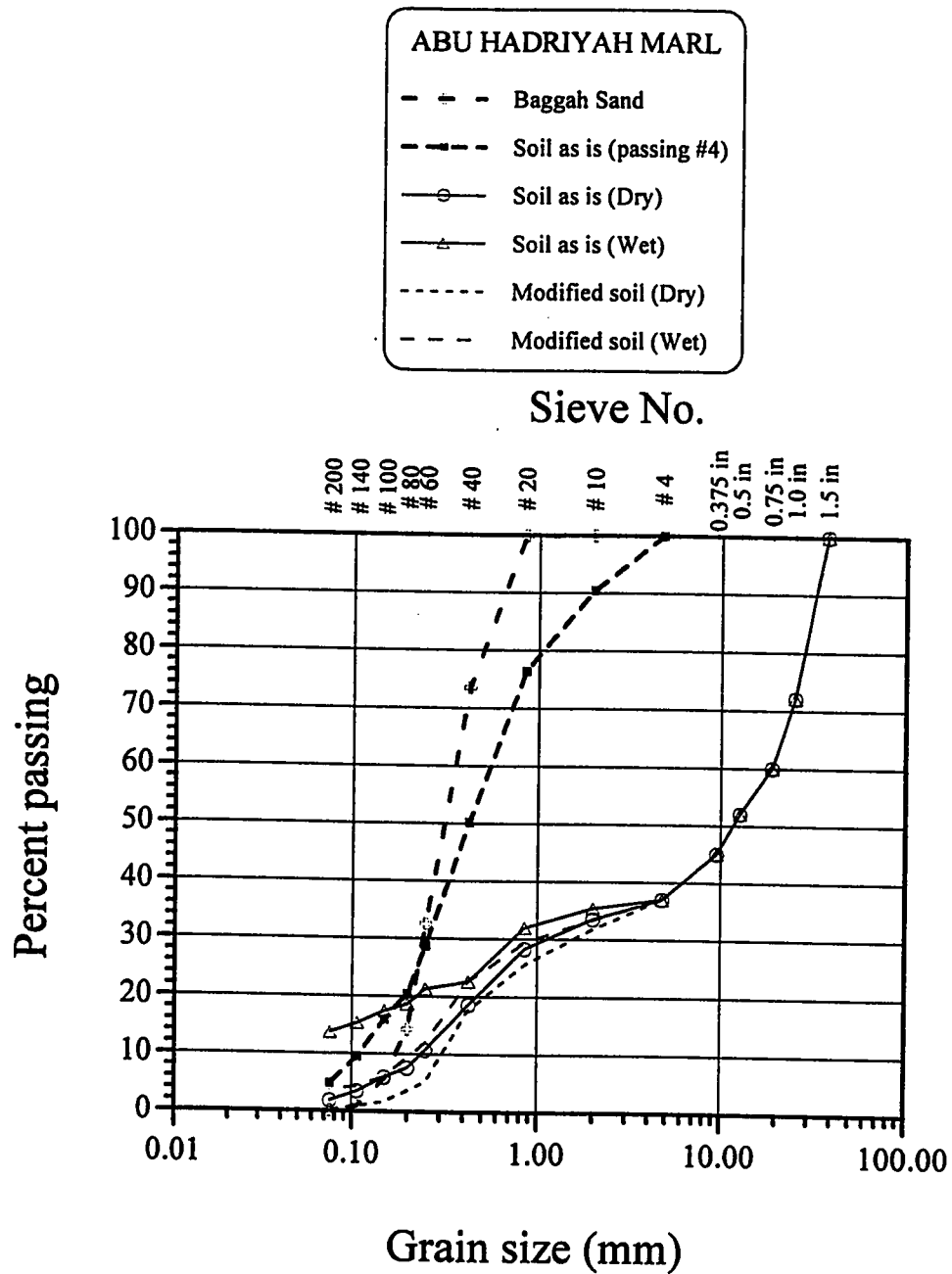


Figure 4.33: Grain-Size Distribution for Baggah Sand, M-ABH9, and M-ABH9a.

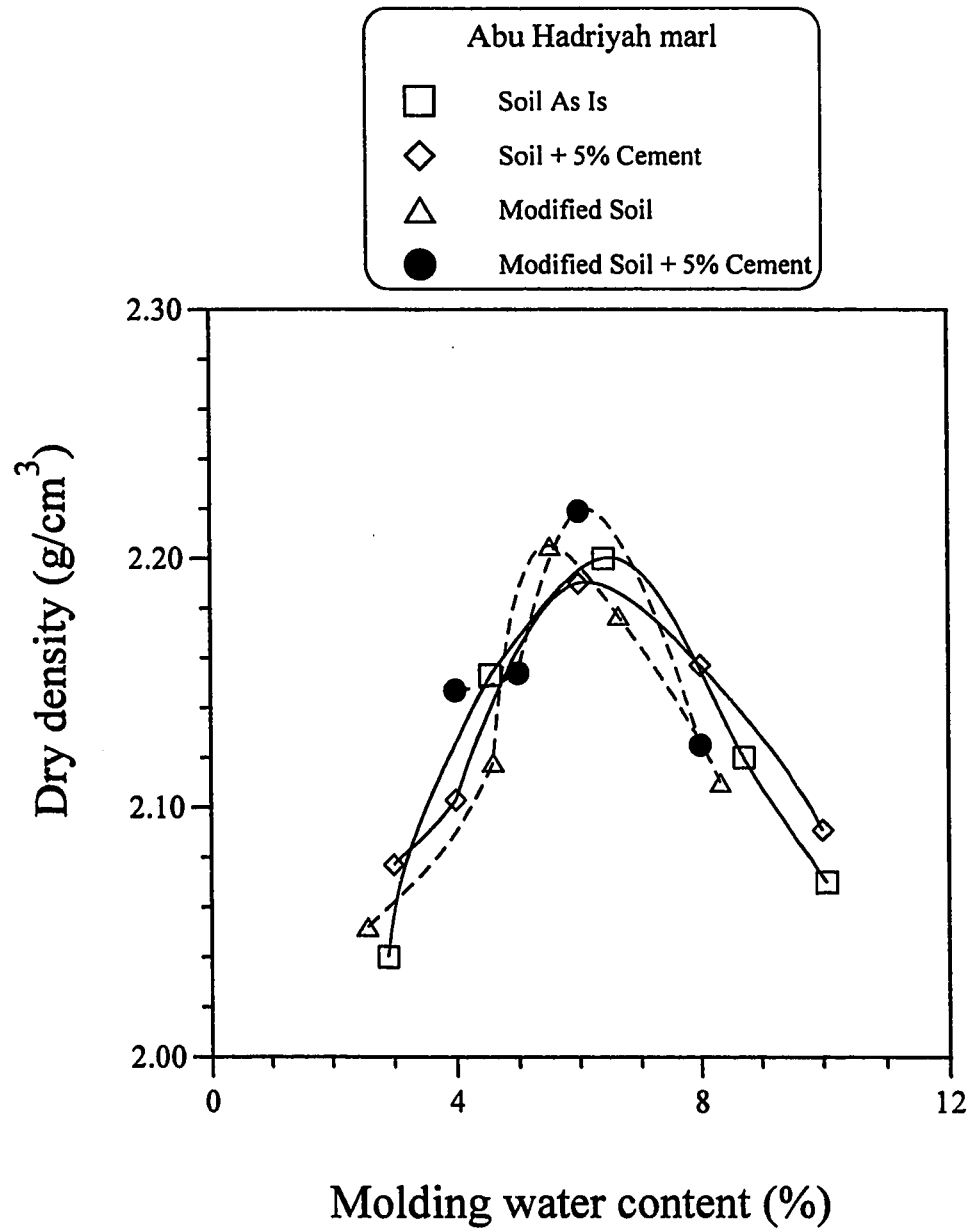


Figure 4.34: Moisture-Density Relationship for M-ABH9 and M-ABH9a.

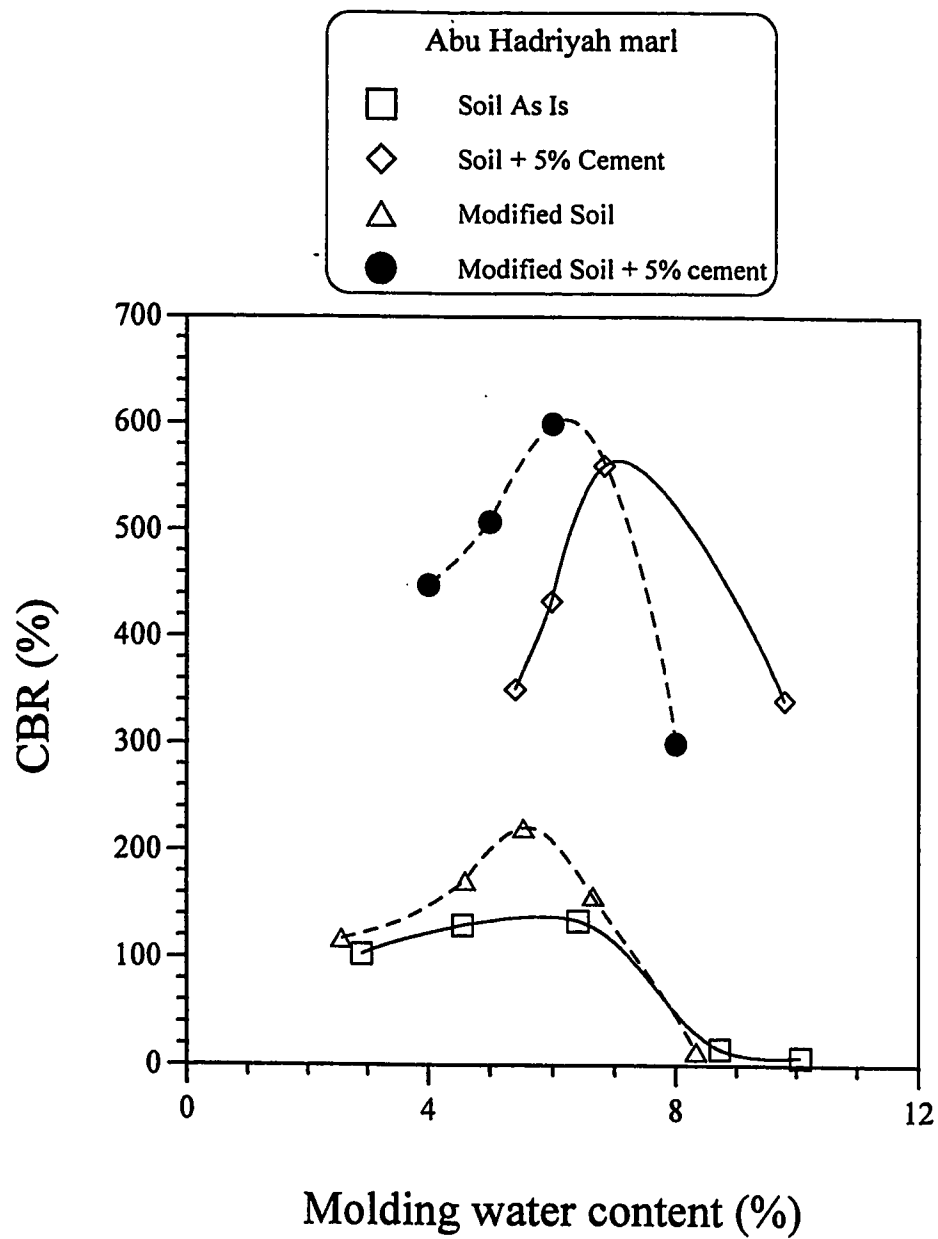


Figure 4.35: Moisture-CBR Relationship for M-ABH9 and M-ABH9a (24 Hours Curing for the Treated Soils).

values of the stabilized soil were taken as decisive in the selection of the appropriate stabilizer. Sulphate resisting (Type V) cement and hydrated lime were used as stabilizers. For both marls (M-ABH9 and M-SHD1), 5% of each additive was used and the variations of unsoaked CBR values, after 7 days of curing, with molding moisture content were plotted in Figs. 4.36 and 4.37. Comparison of the CBR values attained by adding either cement or lime, lead to the conclusion that cement-stabilized marl provides much higher strength.

In addition to strength and stability, economy should also frequently be considered in all engineering projects. Hydrated lime costs about 2.5 times that of Type V portland cement in the local Saudi market. Moreover, cement is effective in treating sandy soils. Since the selected marls are coarse-grained in nature, high values of CBR were the rule rather than the exception. On the other hand, it is well known that lime has no inherent cementitious properties by itself, rather it produces cementitious material only after it reacts with certain constituents present in the soil. These constituents are called pozzolanas such as fly ash, and clay. Owing to the lack of any pozzolanic material in the selected marls, lime did not produce appreciable results. Therefore, sulphate resistant (Type V) cement was selected as the chemical additive to be used for the stabilization of the selected marls. Sulphate resistant cement was selected to provide resistance to the cement stabilized soil against the probable sulphate attack.

4.5.2 Optimization of Cement Content

Once the stabilizing agent has been chosen, an optimum value needs to be determined based on the influencing parameters and construction requirements. Initial

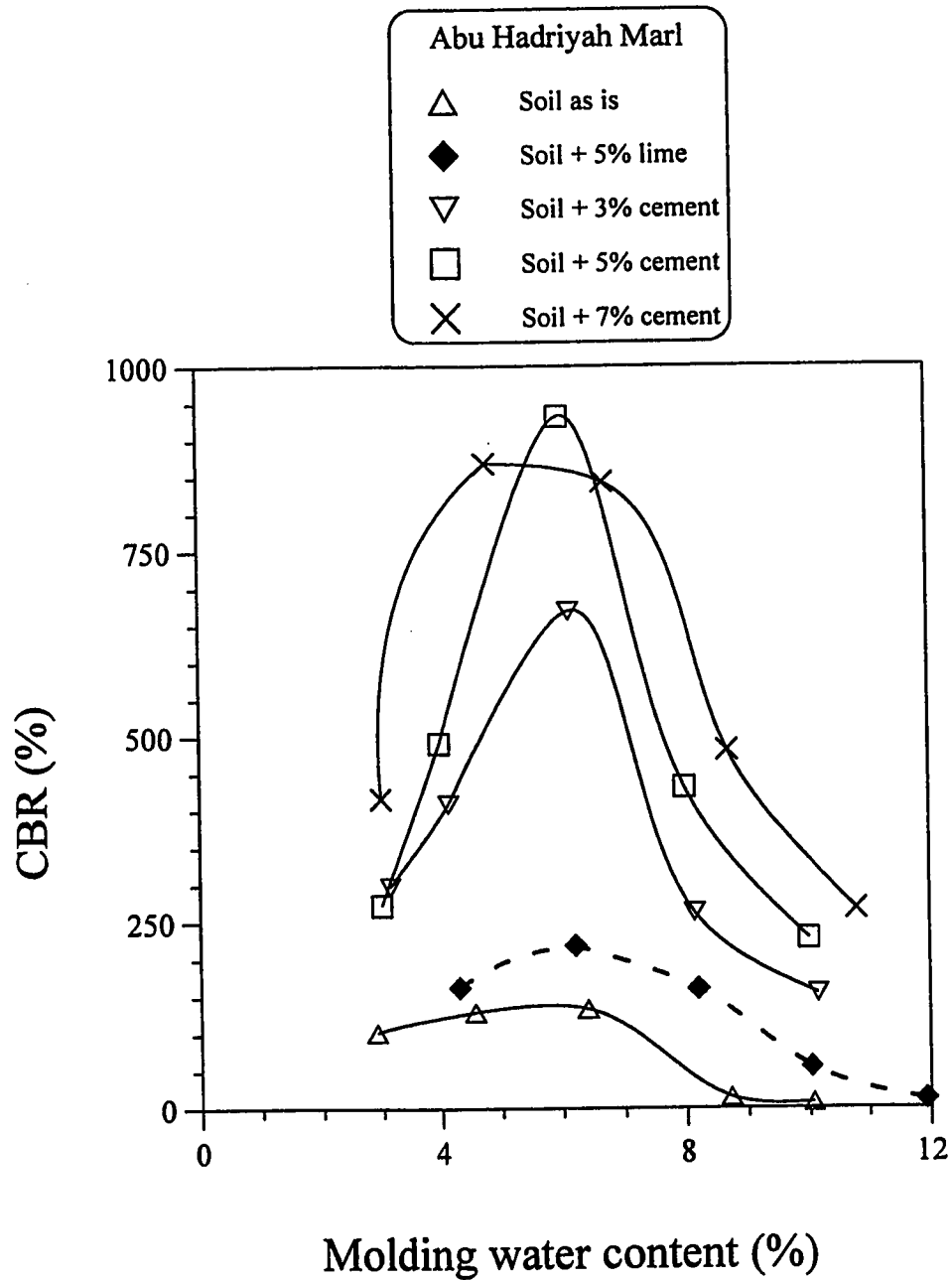


Figure 4.36: Moisture-CBR Relationship for M-ABH9 Stabilized with Cement and Lime (7 Days of Curing).

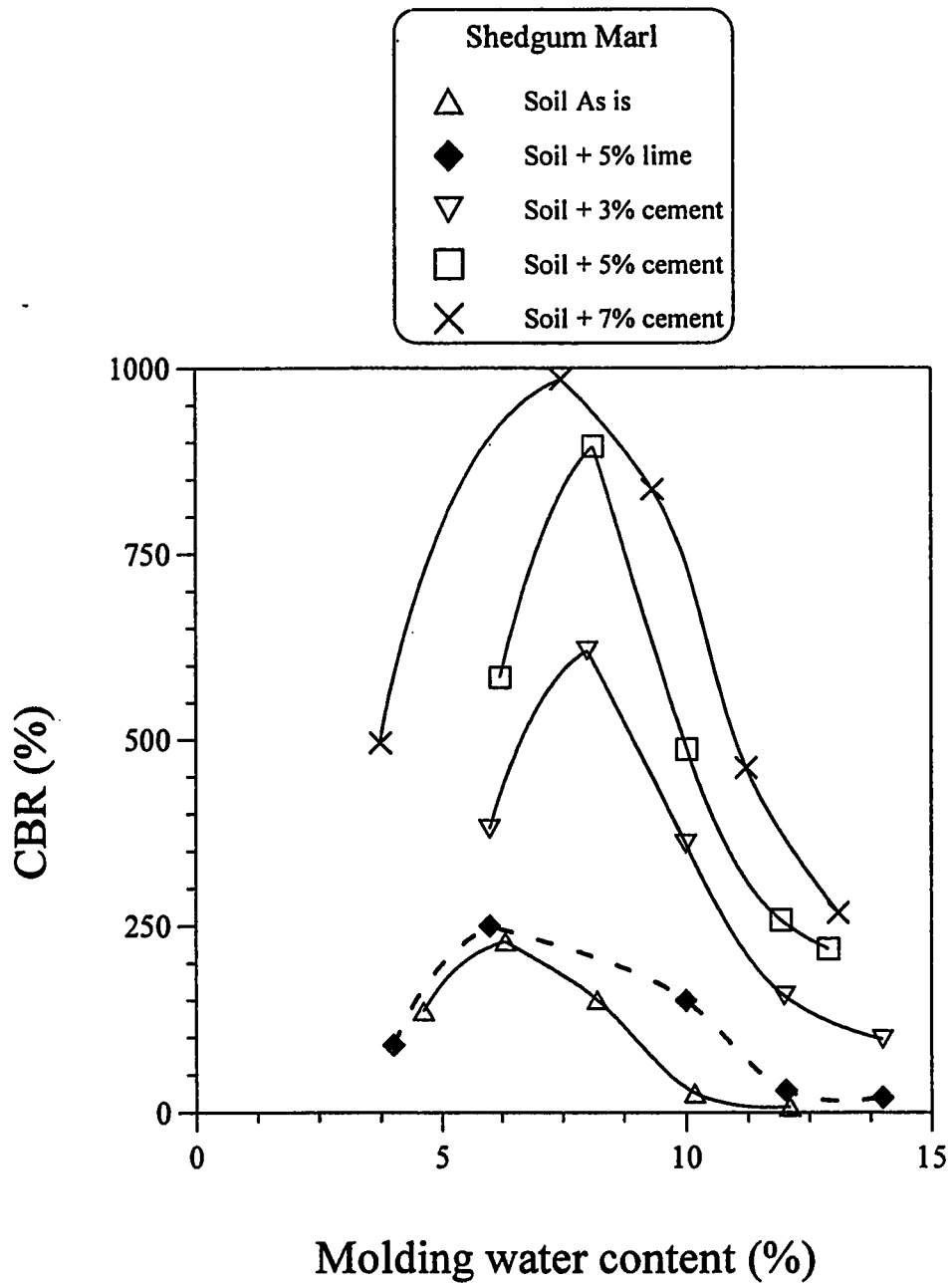


Figure 4.37: Moisture-CBR Relationship for M-SHD1 Stabilized with Cement and Lime (7 Days of Curing).

cement content range was selected using Table 4.6 (ACI Committee 1990). The table provides a range of cement content for the strength and durability for different soil groups. Cement content may vary from 3 to 16% by dry weight of soil depending on the type of soil.

The two selected marls were reconstituted to the gradation specified by the Dammam Municipality for base courses. Both soils are classified as poorly graded gravel (GP) and A-1-a according to the USCS and AASHTO soil classification systems, respectively. Based on this classification, the cement content range of 3-5-7 was adopted for strength as well as for durability requirements, Table 4.6.

Cement Content for Strength Requirements

For each cement percentage, the modified Proctor compaction test was performed on the selected marls to produce moisture-density relationships for the soil-cement as shown in Figs. 4.38 and 4.39. The variation of CBR value, at the optimum moisture content, with cement content is plotted in Fig. 4.40.

Samples for unconfined compressive strength, 100 mm in diameter and 200 mm high, were compacted at the optimum moisture content obtained for 3, 5, and 7% cement contents. Some of the compacted samples were sealed in plastic sheets and others were left exposed. All samples were allowed to cure for 7 days at room temperature (21°C). The variation of unconfined compressive strength (UCS) with cement content is plotted against the cement content in Figs. 4.41 and 4.42, for M-ABH9 and M-SHD1, respectively.

Table 4.6: Typical Cement Requirements for Various Soil Types (ACI Committee, 1990)

AASHTO soil classification	ASTM soil classification	Typical range of cement content requirement *, percent by weight	Typical cement content for moisture-density test**, percent by weight	Typical cement content for durability tests §, percent by weight
A-1-a	GW, GP, GM, SW, SP, SM	3 - 5	5	3 - 5 - 7
A-1-b	GM, GP, SM, SP	5 - 8	6	4 - 6 - 8
A-2	GM, GC, SM, SC	5 - 9	7	5 - 7 - 9
A-3	SP	7 - 11	9	7 - 9 - 11
A-4	CL, ML	7 - 12	10	8 - 10 - 12
A-5	ML, MH, CH	8 - 13	10	8 - 10 - 12
A-6	CL, CH	9 - 15	12	10 - 12 - 14
A-7	MH, CH	10 - 16	13	11 - 13 - 15

* Does not include organic and poorly reacting soils. Also, additional cement may be required for severe exposure conditions such as slope protection

** ASTM D 558

§ ASTM D 559 and D 506

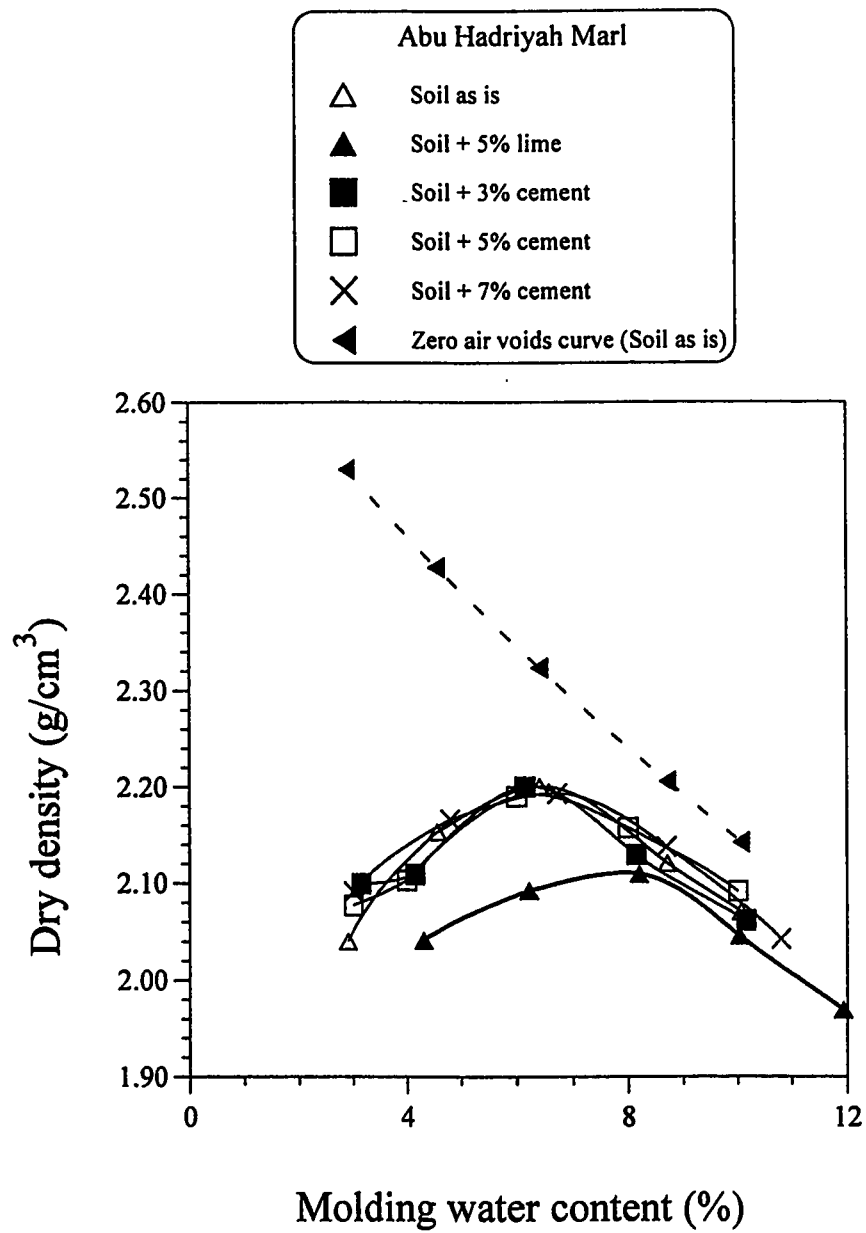


Figure 4.38: Moisture-Density Relationship for M-ABH9 Stabilized with Cement and Lime.

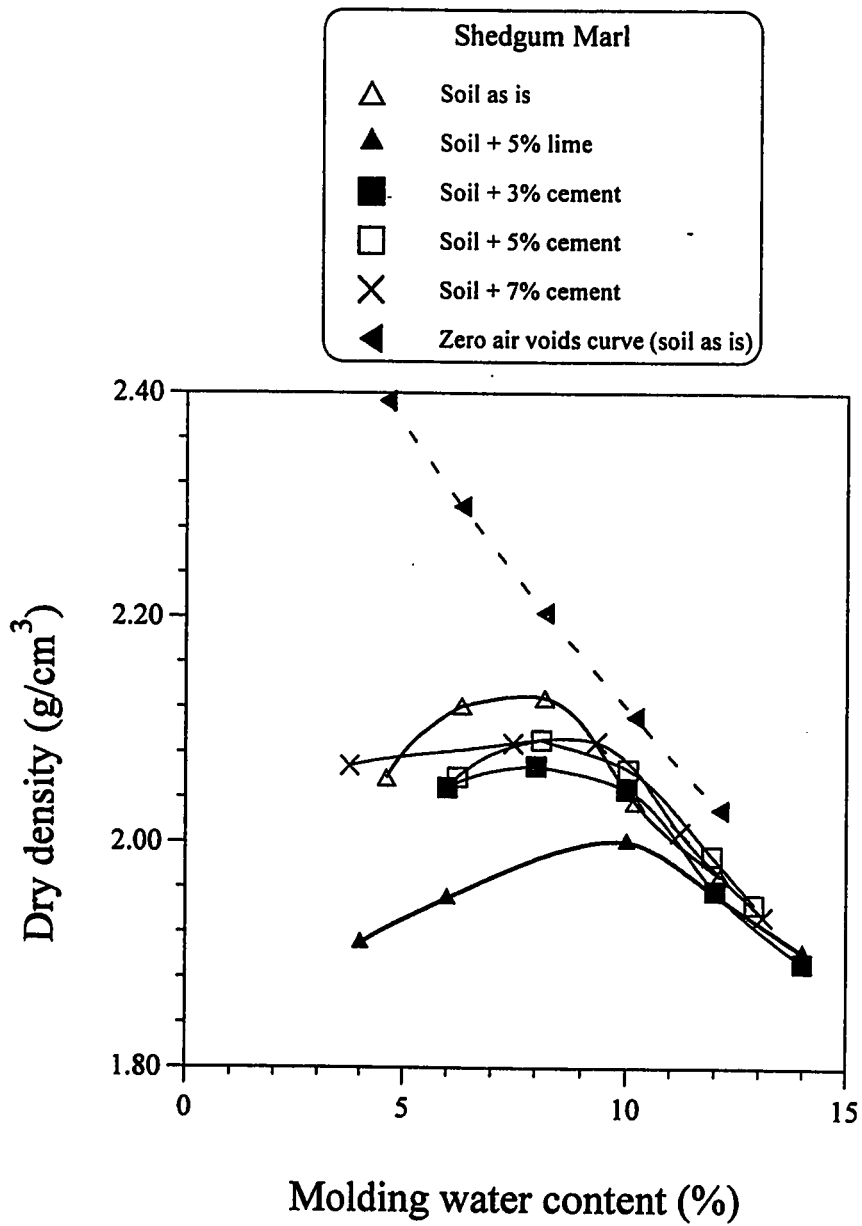


Figure 4.39: Moisture-Density Relationship for M-SHD1 Stabilized with Cement and Lime.

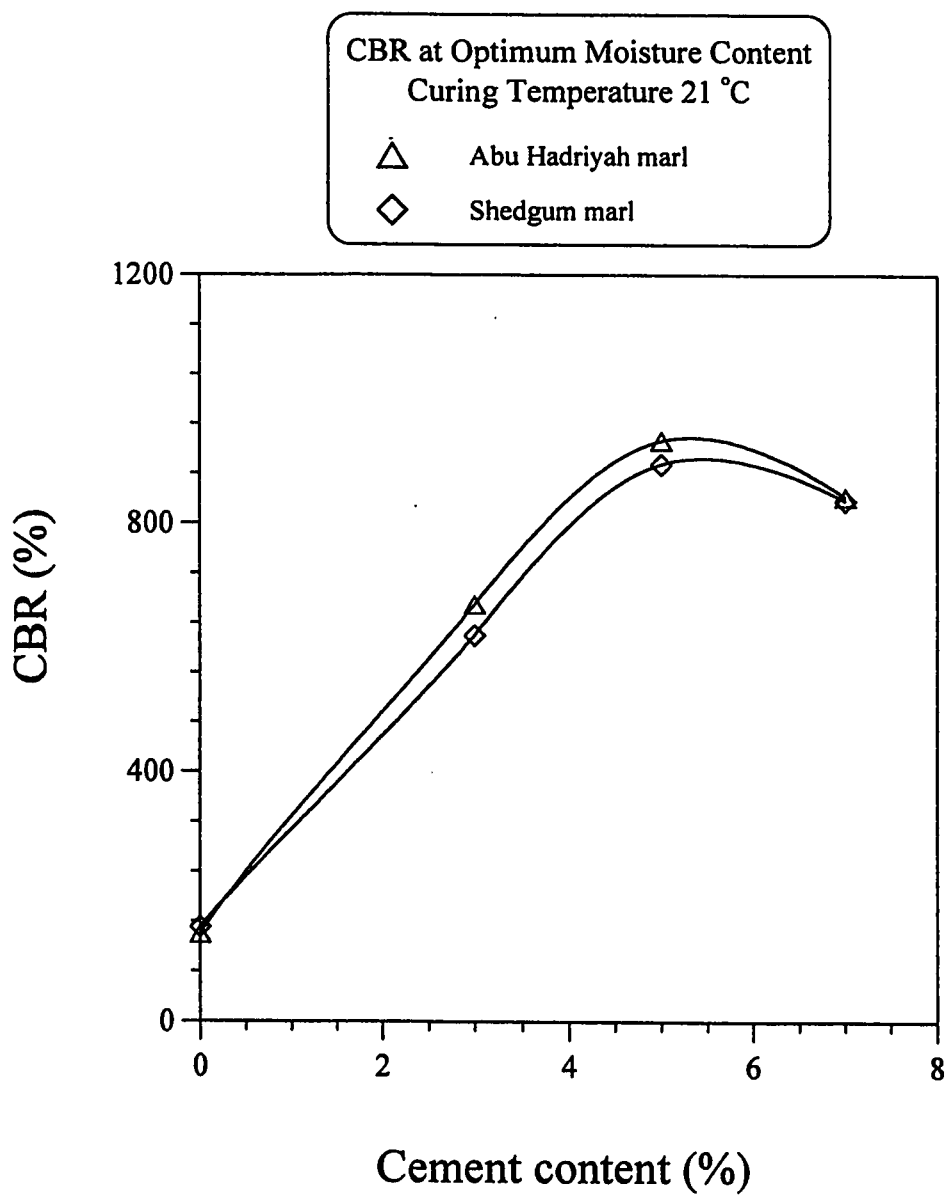


Figure 4.40: Variation of the 7 Days CBR with Cement Content for M-ABH9 and M-SHD1.

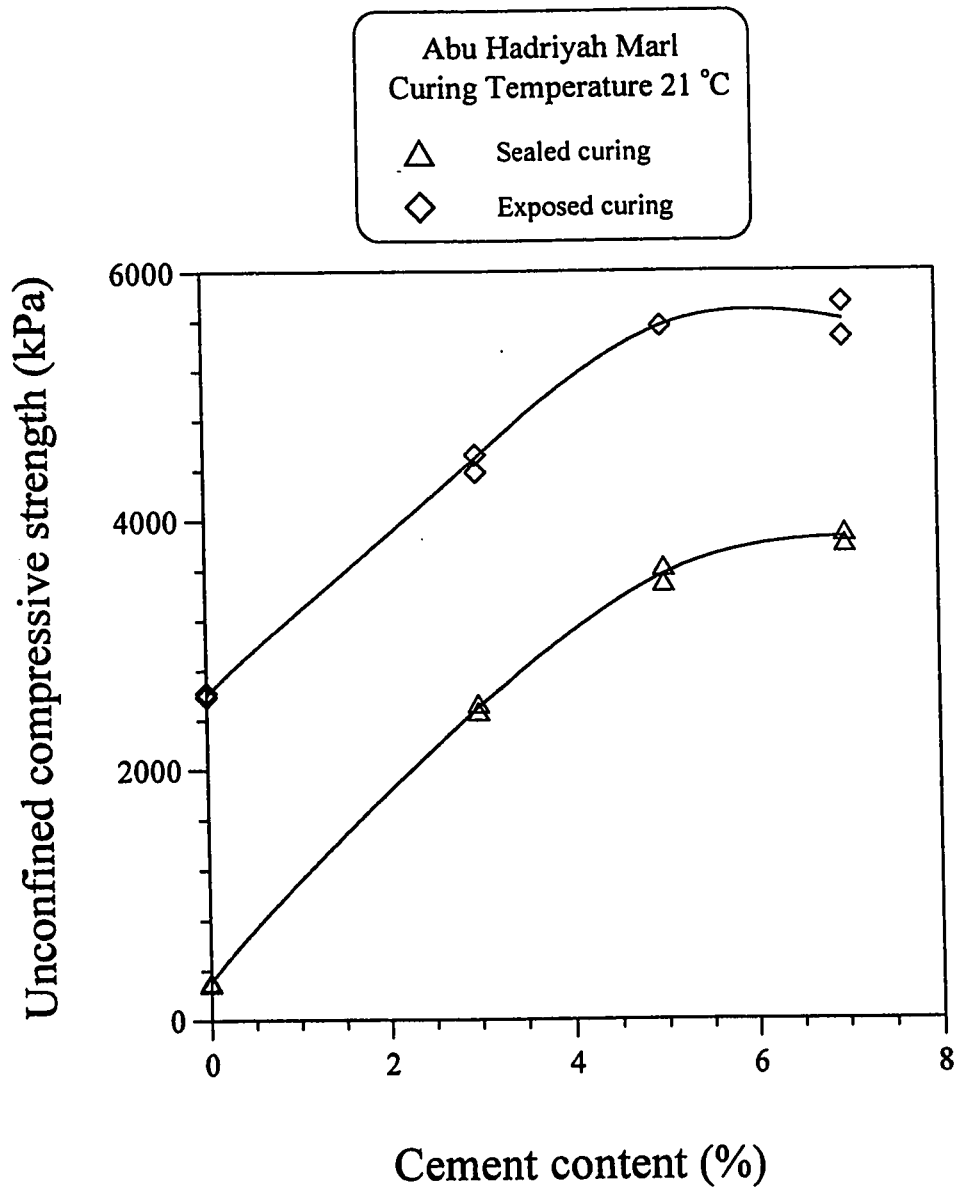


Figure 4.41: Variation of the 7 Days Unconfined Compressive Strength with Cement Content for M-ABH9.

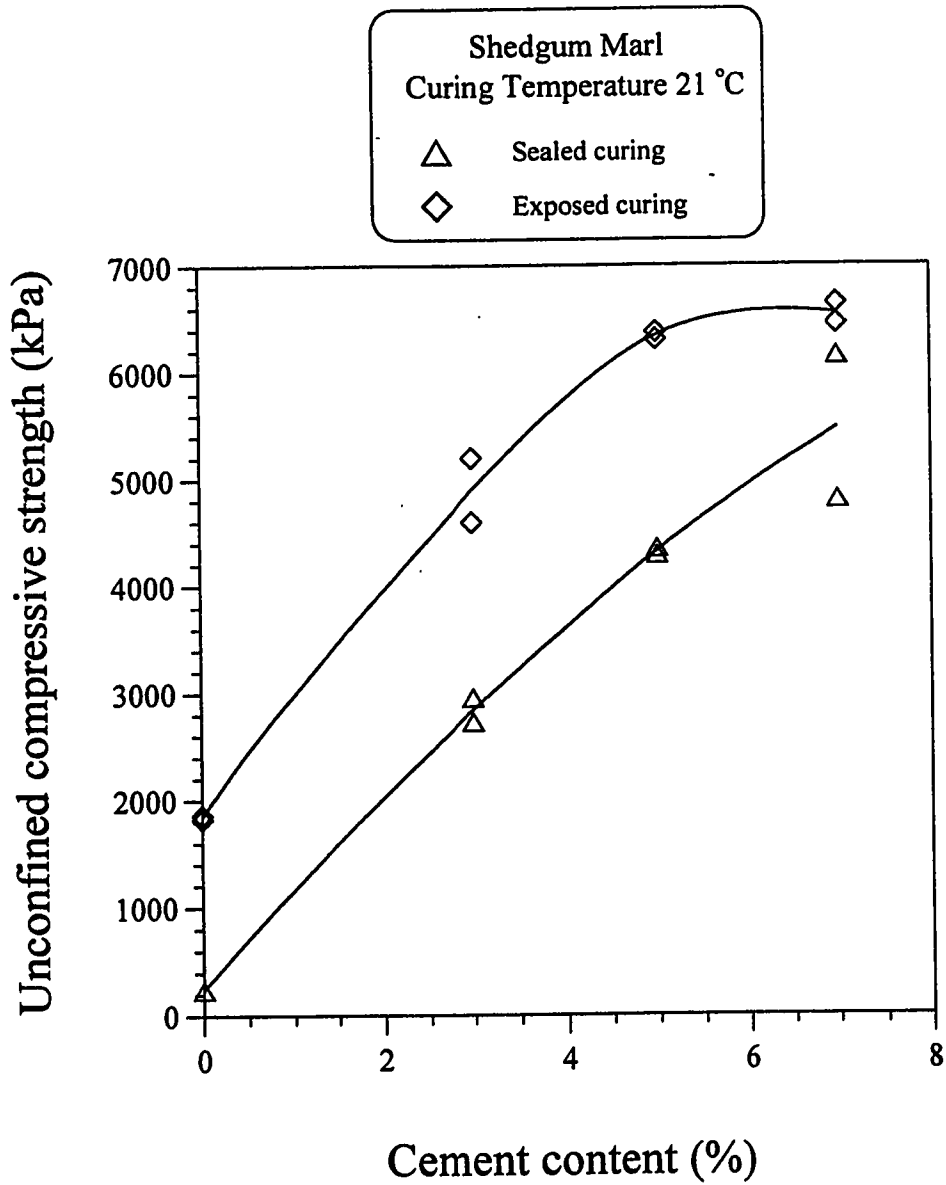


Figure 4.42: Variation of the 7 Days Unconfined Compressive Strength with Cement Content for M-SHD1.

Results reveal that the strength of exposed samples is higher than that of the sealed samples for both marls, irrespective of the cement contents. Moreover, the strength increases as the cement content increases from 0 to 5%. Further increase in cement content did not bring about any significant increase in the UCS value. The UCS for M-ABH9, at a cement content of 5%, was 5558 and 3560 kPa for the exposed and sealed conditions, respectively. The respective values for M-SHD1 were 6343 and 4319 kPa. Since the field curing conditions can be considered somewhat between these two extremes (exposed and sealed), it seems appropriate to take the UCS at any cement content as the average of two extreme values, otherwise, the situation should depend on the exposure conditions in the field.

Based on the above perception, UCS for M-ABH9 and M-SHD1, at 5% cement content, is 4559 and 5331 kPa, respectively. Further, a factor of 1.10 is to be used to get the UCS corrected for a height to diameter (h/d) ratio higher than 1.15 (ACI Committee 1990). It results in UCS values of 5015 kPa (730 psi) and 5864 kPa (850 psi), respectively. According to the ACI Committee 230 report (1990), the minimum UCS specified for the base course construction by the US Army Corps of Engineers (USACE) is 750 psi (5175 kPa). Based on UCS, the required cement content was chosen to be 5% for both marls.

Cement Content for Durability Requirements

The soil-cement fulfilling the minimum strength requirement should also be durable against wetting and drying and freeze and thaw cycles. Depending on the local Saudi environmental conditions, there is no need for investigating the durability against freezing and thawing.

The ASTM standard durability test and modified slake durability test were conducted on soil-cement having 3-5-7% cement contents. Results obtained from the durability tests are shown in Figs 4.43 and 4.44. These figures show the weight loss of the soil-cement specimens for different cement contents for M-ABH9 and M-SHD1, respectively. Maximum weight loss occurs at a cement content of 3% for both soils; it decreases as the cement content increases and almost the same for cement contents of 5 and 7%. The weight loss for a cement content of 5% is 0.5% and 1% for M-ABH9 and M-SHD1, respectively. This is far below the maximum allowable weight loss of 14%, according to Portland Cement Association (PCA) and 9% according to US Army Corps of Engineers (USACE) for soils classified as GP and soils having $PI < 10$, respectively (ACI Committee, 1990). Therefore, a cement content of 5% is considered appropriate not only from strength perspective, but also from durability point of view.

Figs. 4.43 and 4.44 also show that the results obtained by ASTM standard test and slake durability tests are consistent. Moreover, in the ASTM test, application of 3 lb. force during the brushing of samples is always approximate. While in the slake durability test, the abrasion takes place under somewhat controlled conditions, which are reproducible for all the samples. Therefore, the slake durability test can be used for soil-cement specimens instead of the brushing recommended in ASTM durability test. However, further evaluation is required using different soil-cement mixes, preferably with samples having higher losses.

In addition to brushing, volume change and saturation of the soil-cement sam-

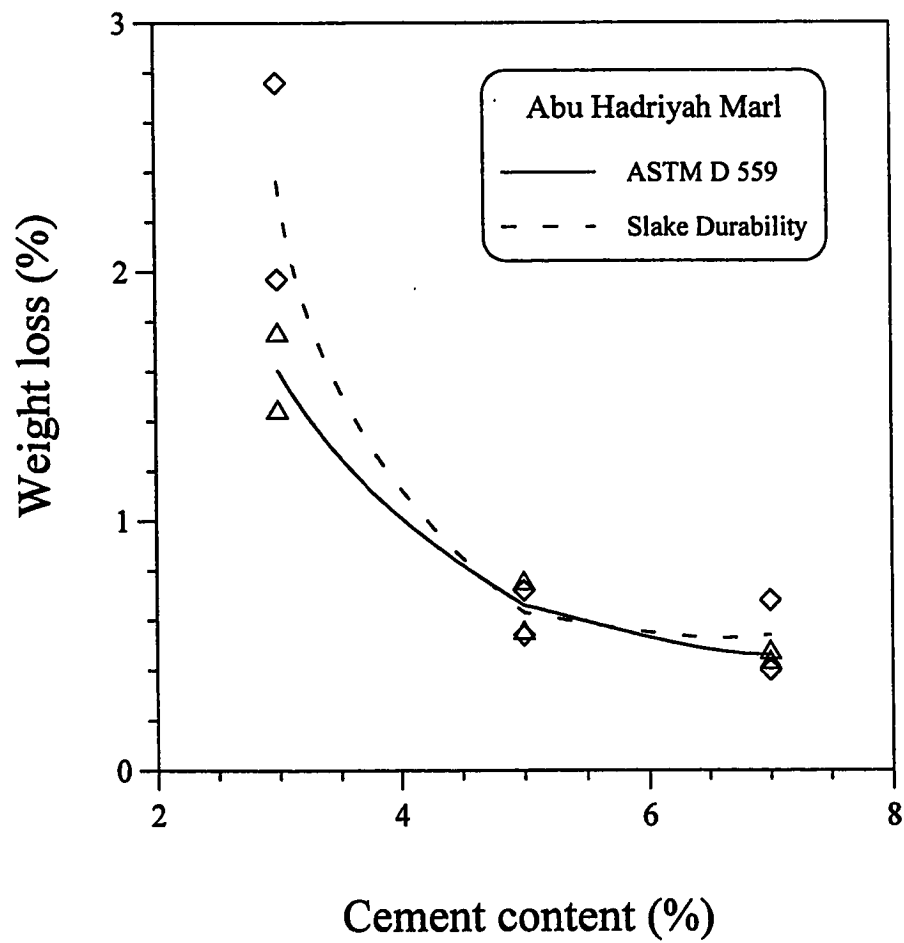


Figure 4.43: Weight Loss For Different Cement Contents During Durability Tests M-ABH9.

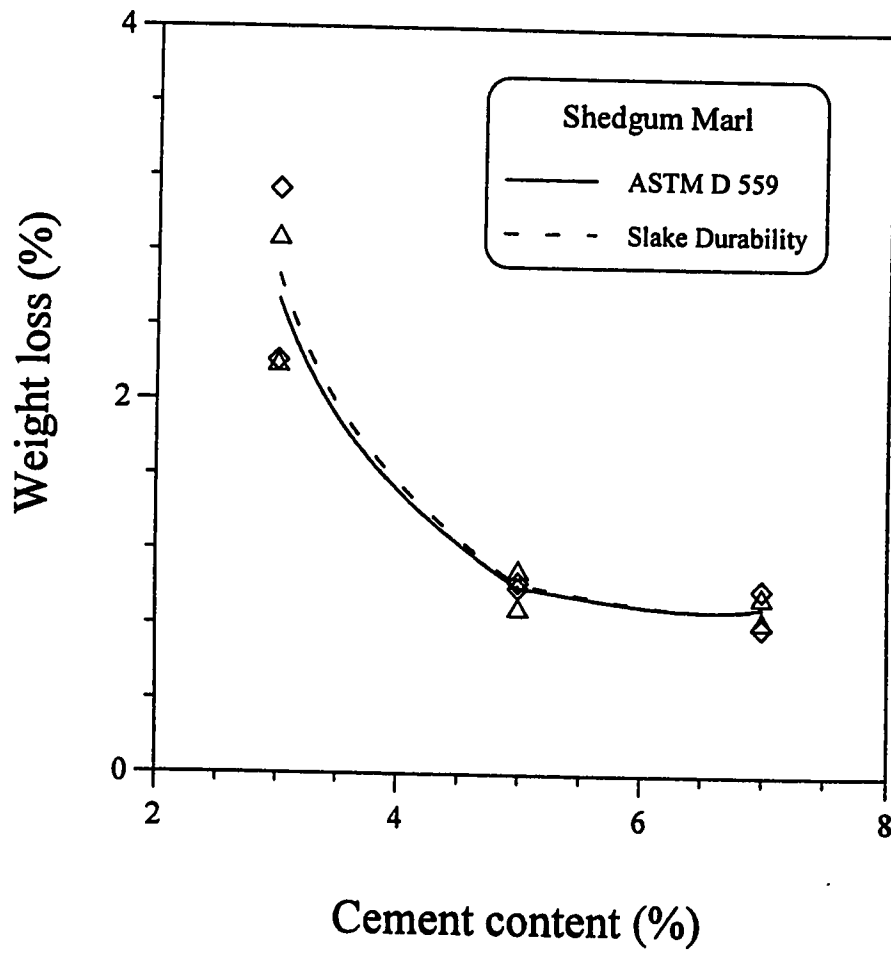


Figure 4.44: Weight Loss For Different Cement Contents During Durability Tests M-SHD1.

ples are a measure of the durability. Soil-cement specimens did not show any volume change during the wet and dry cycles. Moreover, the water content in the soil-cement specimens, at any stage of durability test, did not cause 100% saturation. Thus, both marls, stabilized with cement, proved to be durable for 5% cement content.

4.6 Cement Stabilized Marls

Based on the strength and durability requirements, a cement content of 5% was adopted as the optimum value for stabilizing M-ABH9, M-ABH9a, and M-SHD1 marls. Further investigation was conducted on these cement-stabilized marls to study the effect of other parameters which may alter the behavior of these soils. In particular, the effects of curing regime, delay in compaction, and age on unconfined compressive strength and resilient modulus of the stabilized marls were investigated.

4.6.1 Unconfined Compressive Strength

This test was adopted as a basic test in this research where a comparative study of the effects of different parameters such as curing temperature, curing moisture conditions, delay in compaction, effect of age on the strength gain of the treated marls are to be investigated.

Effect of Curing Temperature

The relative effects of various curing temperatures (21°, 35° and 50°C) were evaluated. The maximum curing temperature was selected on the basis of the average maximum seven days local temperature record as mentioned in Section 3.4.3. Test

results for the selected marls as well as for M-ABH9a treated with 5% cement, are shown in Figs. 4.45 to 4.47, for both sealed and exposed curing.

The general trend obtained from these plots is that the higher the curing temperature, the higher is the UCS. Cement setting and hardening rates tend to increase with an increase in the curing temperature. The initial setting time is decreased by 35 minutes with an increase in temperature of 20°C. In addition, the higher temperature increases the rate of evaporation of water and therefore provides cementation due to the precipitation of carbonates. Table 4.7 shows that the residual moisture contents (moisture content at testing) for the samples cured at higher temperatures were much less than those cured at room temperature; for Abu Hadriyah marl, it is 2% at 21°C and 0.5% at 50°C for the exposed samples. On the other hand, it is not desirable to loose moisture that is needed for cement hydration. The exposed samples exhibited higher strength than the sealed ones for all marl soils except for Shedgum marl at 50°C, as shown in Figs. 4.45 to 4.47. Lower dry density in the case of Shedgum marl was responsible for higher evaporation loss in the case of exposed samples at higher temperatures. This results in unavailability of sufficient water for cement hydration thereby leading to a lower strength value. The lower dry density of the samples compacted after certain delay also affected the strength of exposed samples as shown in Fig. 4.48. While in the case of Abu Hadriyah marl, the relatively high dry density prevented some water from evaporation. That water was used for cement hydration, and thus resulting in higher strength.

Loss of moisture at any particular temperature is also controlled by some other

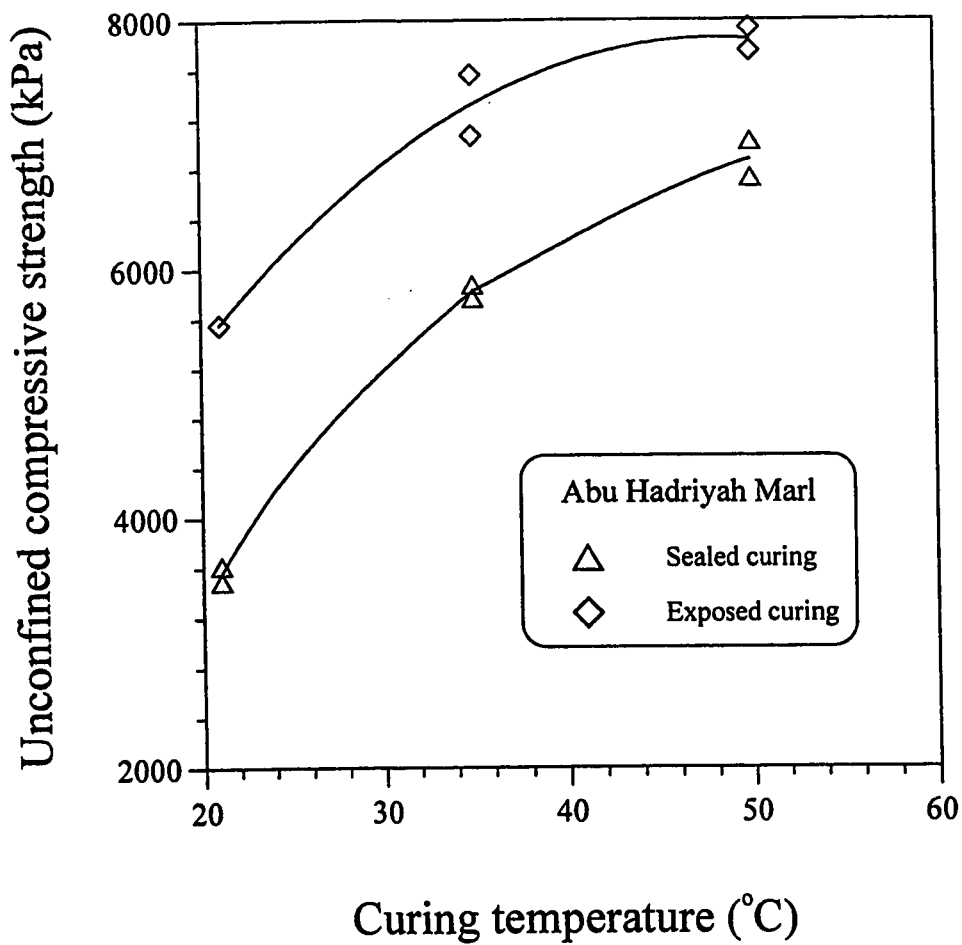


Figure 4.45: Variation of the 7 Days Unconfined Compressive Strength with Curing Temperature for M-ABH9 Treated with 5% cement.

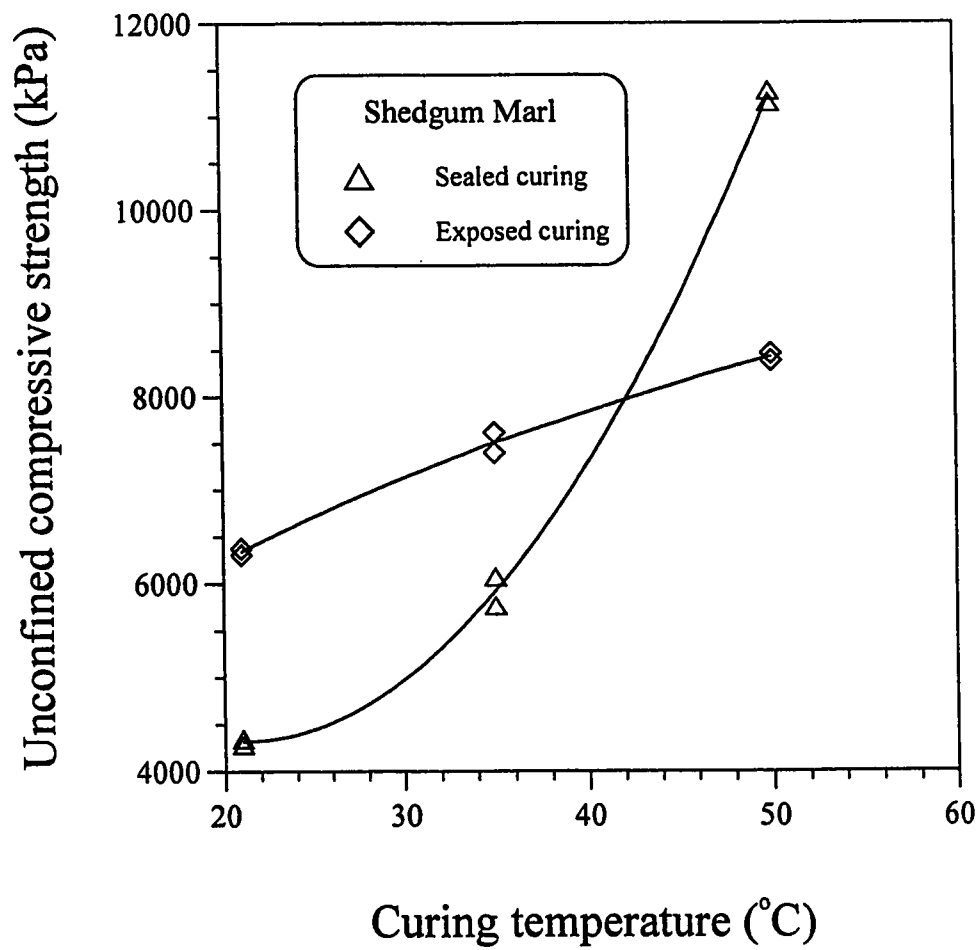


Figure 4.46: Variation of the 7 Days Unconfined Compressive Strength with Curing Temperature for M-SHD1 Treated with 5% cement.

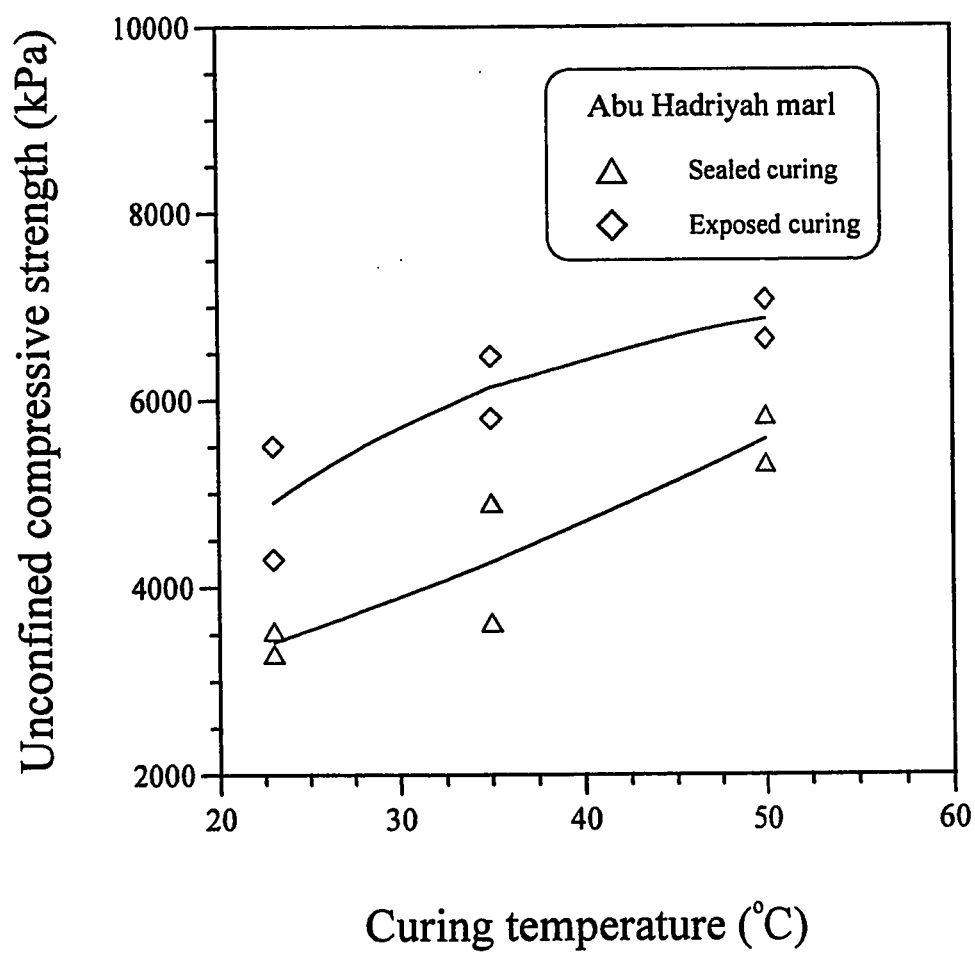


Figure 4.47: Variation of the 7 Days Unconfined Compressive Strength with Curing Temperature for M-ABH9a Treated with 5% cement.

Table 4.7: Residual Moisture Contents for Stabilized and Untreated Samples of M-ABH9 and M-SHD1 after 7 Days of Curing

Parameters				Marl					
Cement Content (%)	Curing Temperature (°C)	Delay (hours)	Age (days)	M-ABH9			M-SHD1		
				cmc *	rmc ^s (%)		cmc *	rmc ^s (%)	
				(%)	Sealed	Exposed	(%)	Sealed	Exposed
0	21	0	7	6.50	6.15	1.25	8.00	8.00	0.90
3	21	0	7	6.30	6.15	2.00	8.00	7.50	1.60
5	21	0	7	6.30	5.80	2.00	8.00	6.75	1.50
7	21	0	7	6.30	5.92	2.00	8.00	7.35	1.90
5	35	0	7	6.30	4.30	1.10	8.00	6.00	0.80
5	50	0	7	6.30	3.60	0.50	8.00	4.00	0.50
5	21	0	7	6.30	5.80	2.00	8.00	6.75	1.50
5	21	1	7	6.00	5.40	1.80	7.80	7.50	2.10
5	21	2	7	5.75	5.40	1.80	7.70	7.50	2.00
5	21	4	7	5.60	5.40	---	7.60	7.50	---
5	21	0	1	6.30	5.50	3.00	---	---	---
5	21	0	3	6.30	5.60	2.50	---	---	---
5	21	0	7	6.30	5.80	2.00	---	---	---
5	21	0	14	6.30	5.00	1.50	---	---	---
5	21	0	28	6.30	4.80	1.20	---	---	---

* cmc = compaction moisture content

^s rmc = residual moisture content

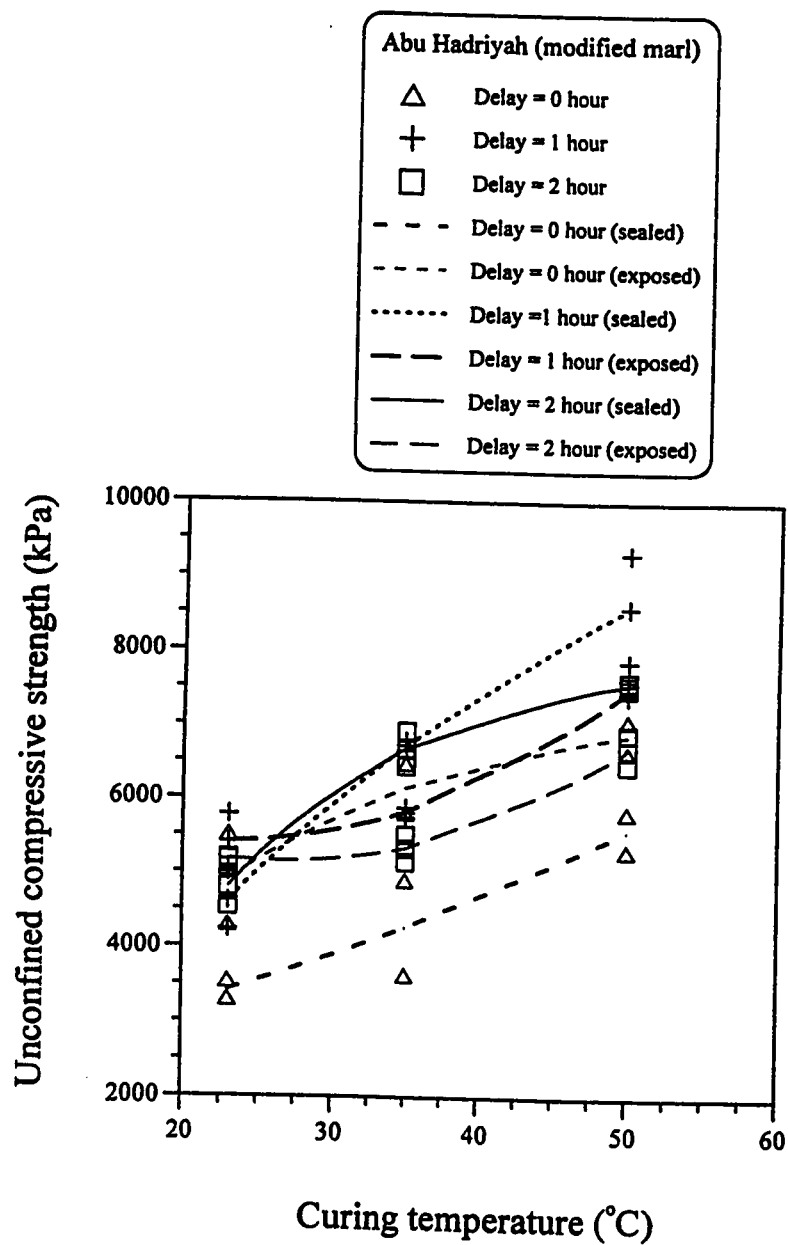


Figure 4.48: Variation of the 7 Days Unconfined Compressive Strength with Curing Temperature for M-ABH9a Samples Compacted after a Delay of 1, 2 and 4 Hours.

factors including the dry density and the fabric of the soil. Lower dry density and open fabric result in higher evaporation loss. The curves in Fig. 4.48 show the effect of curing temperatures and compaction delay on the strength of the modified Abu Hadriyah marl samples. After mixing, the compaction of these samples was delayed for specified periods. This delay causes a decrease in the dry density. The curves clearly indicate that the exposed specimens resulted in higher strengths for all the curing temperatures when compaction was not delayed. On the other hand, the delay in compaction gave higher strength for sealed samples at higher curing temperatures only, when compared to the exposed samples. The lower density for the delayed samples is responsible for the loss of water at higher curing temperatures, therefore, retardation of hydration occurs, leading to a reduction in strength. Higher dry density of the samples that had no delay, produced higher strength for the exposed samples at all temperatures.

Similar conclusions for the effect of curing temperature on the strength of cement-stabilized soils have been reached by other investigators. Bhatia (1967) found that the curing temperature has very pronounced effect on the strength of soil-cement mixtures. He stated that the results obtained by Nurse (1951) indicate that the strength of soil-cement mixtures increased as the curing temperature increased from 0°C upward. The increase in strength continued up to a certain curing temperature that is dependent on the type of soil. Further increase in temperature caused a decrease in strength. For organic sands and silt-clay, the optimum temperature was found to be 45°C and 60°C, respectively, while for clay, the strength was increasing up to a temperature of 60°C. Bhatia (1967) found that soil-cement samples show 2 to 2.5 % increase in strength for each one °C rise

in temperature, for samples cured for 7 days, and for a temperature range of 25 to 50°C. He also found that high curing temperature can give the 7 day strength in just 24 hours. Soil-cement construction in spring or summer will have higher strength than the one constructed in autumn or winter.

Great care is required when interpreting the laboratory curing temperature results for field conditions. This is mainly due to the entirely different boundary conditions in the field compared to the laboratory. In the case of the laboratory 100 mm diameter cylindrical samples, the whole soil sample is affected by the laboratory temperature. The temperature at the center of the sample is not very different from that at the surface. This produces a uniform effect. On the other hand, the temperature in the field, at few inches from the surface will be different from that in deeper soils. The gradient depends on the thermal conductivity of the soil and on the local temperature. The thermal conductivity depends on type, moisture conditions, and dry density of soil. In general, the curing conditions of the whole soil layer will not be uniform and, therefore, the strength of the layer will probably vary with thickness. In addition, the field temperature varies continuously during the day and night while laboratory samples are kept at a constant temperature.

Effect of Moisture Conditions

When using cement-stabilized materials, curing for a specific time, under certain conditions, is a basic requirement. Curing ensures the availability of water for the hydration of cement throughout the curing period. Water sprinkling and bitumen coating are commonly used. The curing regimes adopted in the laboratory

were chosen to simulate the two extreme curing conditions in the field, namely sealed and exposed regimes. In the sealed curing, the samples were completely wrapped with several layers of plastic sheets right after their extrusion from the mold; while in the exposed curing, the samples were kept exposed without any wrapping. The sealed curing condition adopted in the laboratory simulates the proper curing procedures in the field. The field curing methods do inhibit the moisture loss, but they do not ensure complete control. On the other hand, complete sealing of laboratory soil-cement samples using plastic sheets ensures no moisture loss. The exposed curing conditions, in which no sealing was used, simulates the field if no curing method is used to prevent moisture loss. This is usually due to negligence or lack of knowledge of importance of curing by some of the contractors.

Water required for complete hydration of cement is about one fourth of its weight (Ingles and Metcalf, 1972). For instance, for 5% cement content, 1.25% water of the soil weight is required for complete hydration. In the case of selected marls, the optimum moisture content used in the preparation of soil-cement mixtures is well above the required amount for cement hydration. However, exposed curing conditions do not prevent water evaporation from the soil-cement samples.

The loss of water from the soil-cement samples has both negative and positive effects. The negative effect is the unavailability of water for the hydration of cement. The positive effect is the carbonate cementation upon evaporation of water. Therefore, exposed curing conditions can be helpful if the moisture loss does not cause depletion of the least moisture required for the hydration of cement. The moisture loss during exposed curing depends on temperature and relative

humidity. The relative humidity in the laboratory is kept around 50% under the air-conditioned environment, while in the field, it varies from 30% up to 100%. Locally, in the Eastern Province, the relative humidity during the summer session remains well above 50%, and very close to 90%. A relative humidity of 100% at any temperature has the same effect as that of the sealed conditions, i.e., little evaporation of water occurs. On the other hand, lower relative humidity results in greater loss of moisture. Therefore, the results obtained under the laboratory curing conditions should be interpreted after considering the conditions prevailing in the field.

Effect of Delay in Compaction

The effects of delay in compaction was investigated by allowing some delay after mixing. The dry density, UCS, and resilient modulus were used to study the effect of delay in compaction for the three soil types (M-ABH9, M-ABH9a, and M-SHD1). Delay in compaction of 1/2, 1, 2, and 4 hours was imposed. Samples compacted after the specified delay were exposed to different curing conditions. M-ABH9a samples were allowed to cure under sealed and exposed conditions and under different temperatures. While the samples for the other two selected marls were allowed to cure under sealed and exposed conditions at room temperature only. The results presented in Figs. 4.49 to 4.53 report the effect of delay on the strength of samples under the different curing conditions.

The results in Figs. 4.49 to 4.53 show that at room temperature, the UCS of sealed samples is increasing within the first hour of compaction delay. Further delay, however, resulted in a decrease in strength. This behavior is typical for all

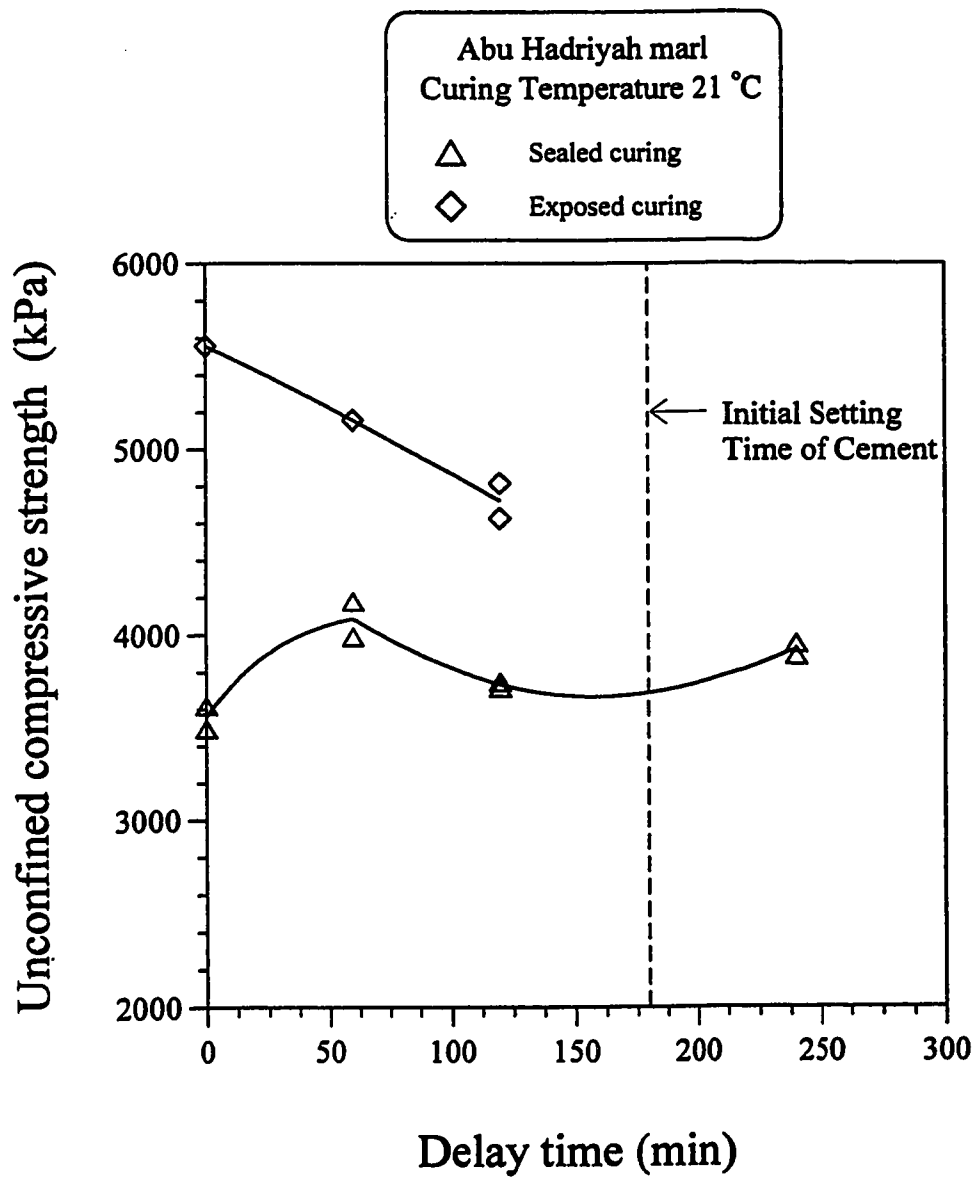


Figure 4.49: Variation of the 7 Days Unconfined Compressive Strength with Delay in Compaction for M-ABH9.

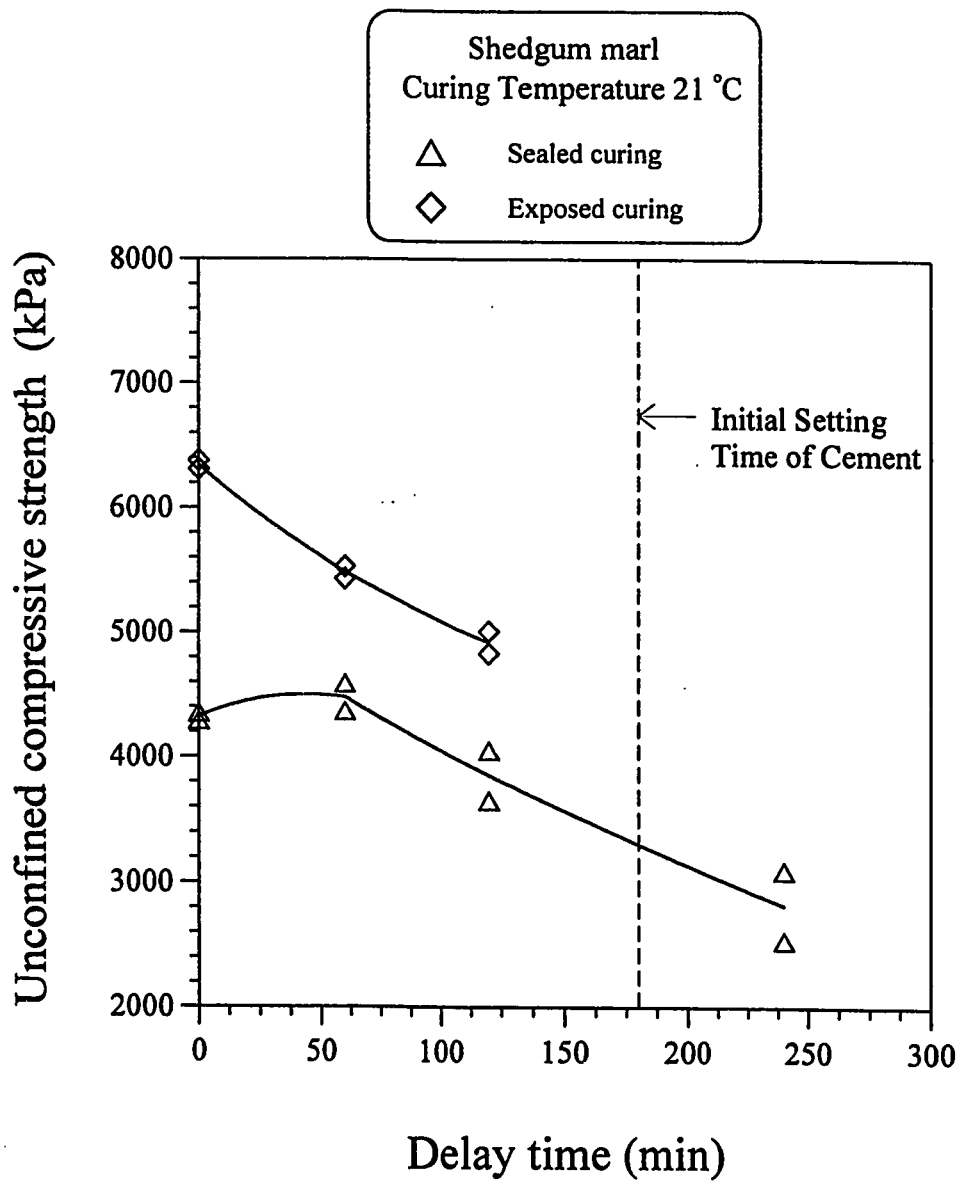


Figure 4.50: Variation of the 7 Days Unconfined Compressive Strength with Delay in Compaction for M-SHD1.

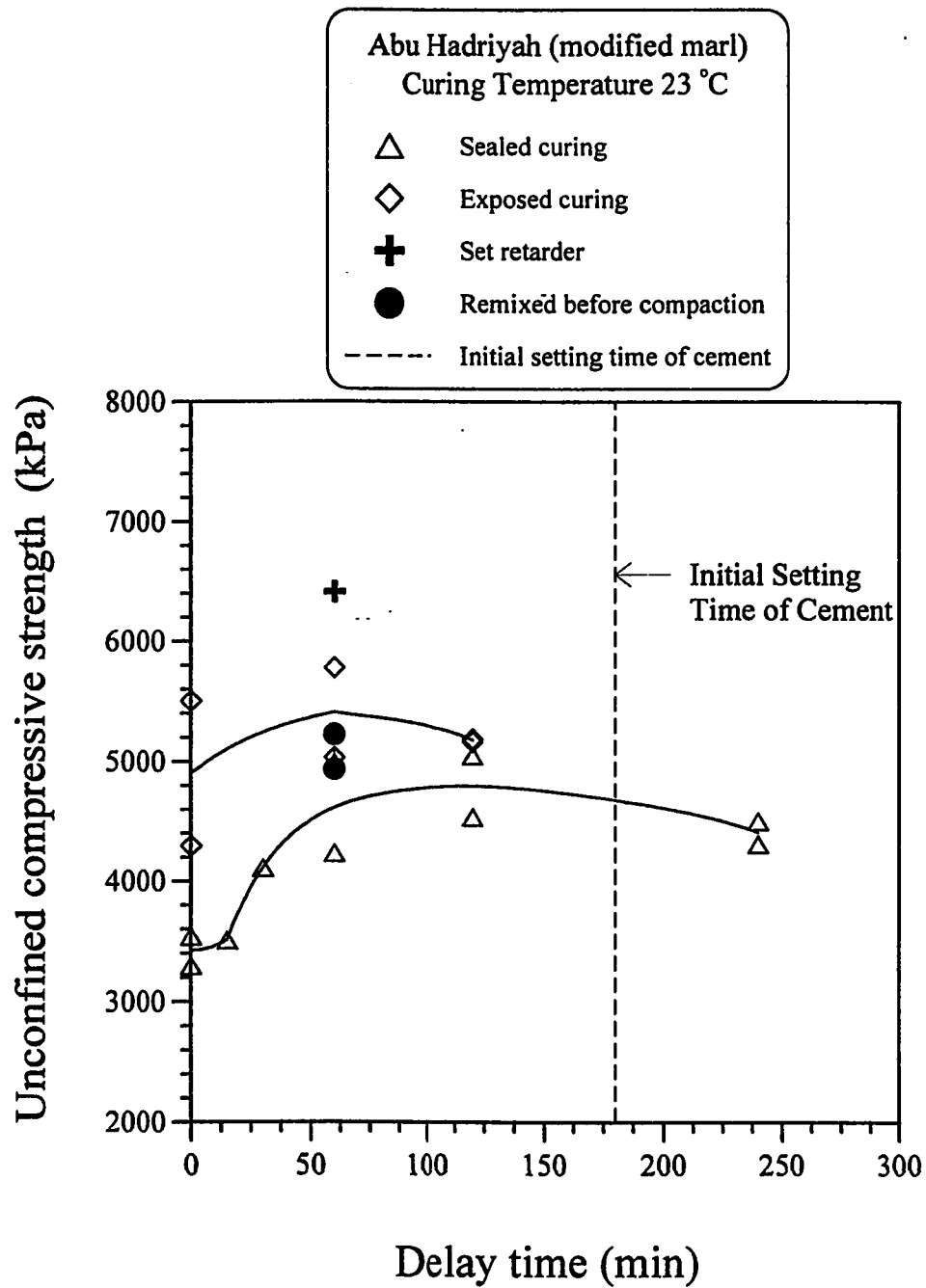


Figure 4.51: Variation of the 7 Days Unconfined Compressive Strength with Delay in Compaction for M-ABH9a.

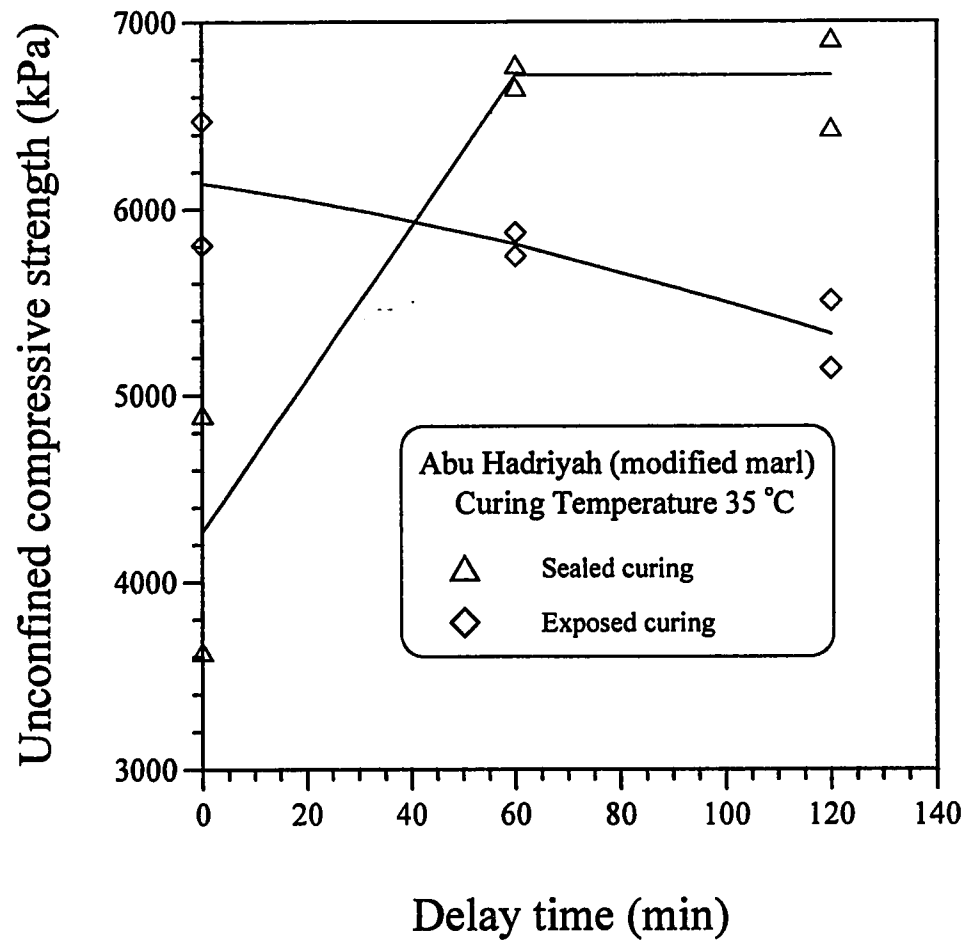


Figure 4.52: Variation of the 7 Days Unconfined Compressive Strength with Delay in Compaction for M-ABH9a.

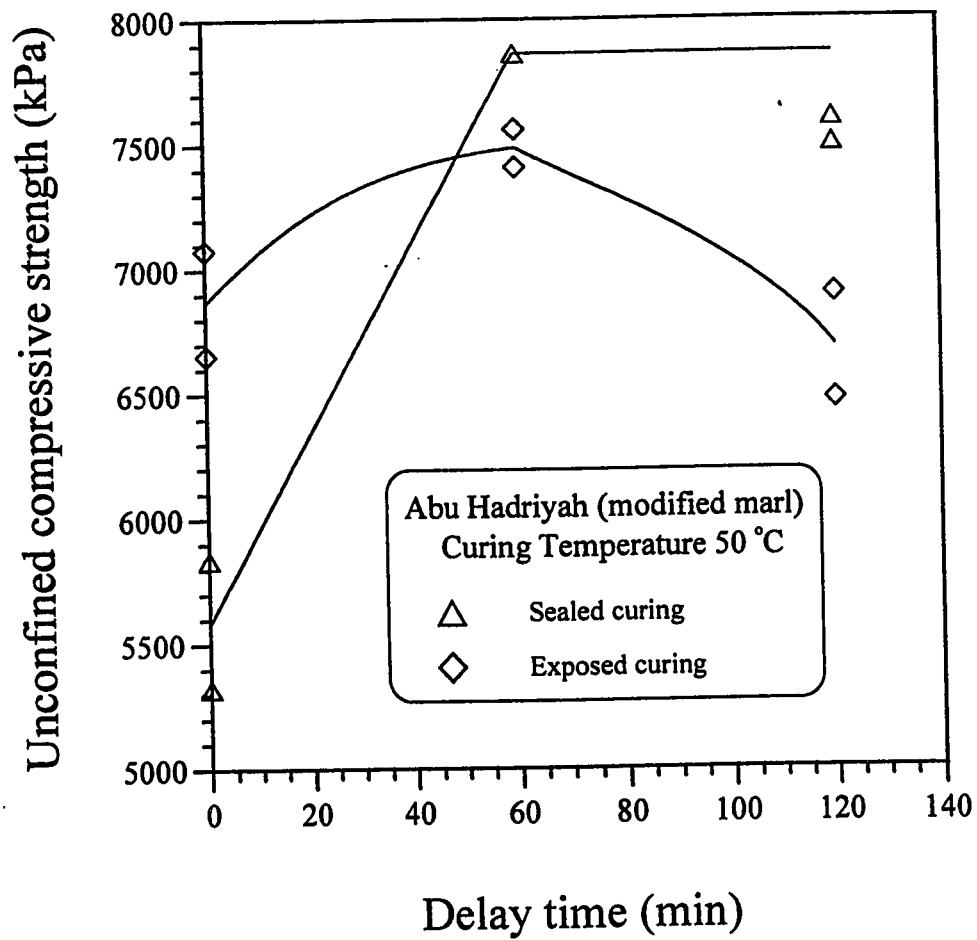


Figure 4.53: Variation of the 7 Days Unconfined Compressive Strength with Delay in Compaction for M-ABH9a.

the marls tested in this investigation (M-ABH9, M-SHD1, and M-ABH9a). This anomalous behavior of strength increase for the sealed samples up to a certain delay is attributed to several factors such as higher frictional resistance due to coarse soil-cement clods, and normal cement hydration for one hour delayed compacted samples. Although there was a decrease in density with delay, some positive phenomena also occur.

Within a delay of one hour, the fines formed clods of larger sizes, resulting in somewhat a coarser material. It was observed by Arman and Saifan (1967) that for the soil-cement samples that had been compacted immediately after mixing, the grains loosened by brushing during the durability testing, had approximately the same size and shape of the original soil particles. While the samples compacted after some delay after mixing, the grains loosened were considerably larger in size, indicating that the groups of individual particles were cemented together before compaction. Therefore, these large-sized particles, or more accurately aggregated particles, contributed towards the higher strength through a better interlocking. In addition, Table 4.7 shows that the moisture consumed (during the 7 days of curing) by the sealed samples for one hour delay is the same as that by the samples which were not given any delay. This shows that the hydration of cement after one hour of delay will continue at almost a normal rate; which is not the case if delay is more than one hour. For samples with more than one hour delay, more moisture was consumed during the delay time and thus less water was available during compaction. Moreover, when delay in compaction is more than one hour, there is formation of a poor cement bond. This resulted due the approaching of the initial setting time of cement.

In case of marly soils, the carbonate particles are susceptible to dissolution in the pore water. The dissolved carbonates will precipitate at the particle contacts either due to evaporation of water or decrease in partial pressure of dissolved carbon dioxide. In case of sealed samples, the latter factor dominates while for the exposed samples both factors are contributing simultaneously. The suction present in the soil pores may cause dissolved carbon dioxide to be liberated from water, resulting in the precipitation of carbonates. Hence, the cementation provided by the precipitated carbonates augmented the strength gained by the soil cement bonding. Thus, the positive factors will dominate until a delay of one hour, after which further delay contribute negatively. This is mainly due to the significant reduction in dry density, and the initial setting time (IST) that will occur within the initial 3 hours of hydration. Soil-cement mixtures compacted close to or after the IST of cement will not develop a good quality cementitious material. This factor will dominate when the delay is more than one hour, and was responsible for the decrease in strength of the marl soil.

One of the consequence of delay in compaction was the reduction in dry density, as shown in Figs. 4.54 to 4.56. There was a continuous decrease in the dry density with the delay and the decrease could be as much as 5%. When the cement comes into contact with water, it starts hydrating. Although there is no intimate contact between the cement and soil particles, soil-cement clods will form even in the loose state. The finer particles agglomerate to form relatively large and strong clods. During compaction, these large agglomerates resist densification, thus resulting in lower dry densities. Moreover, it has been noticed that

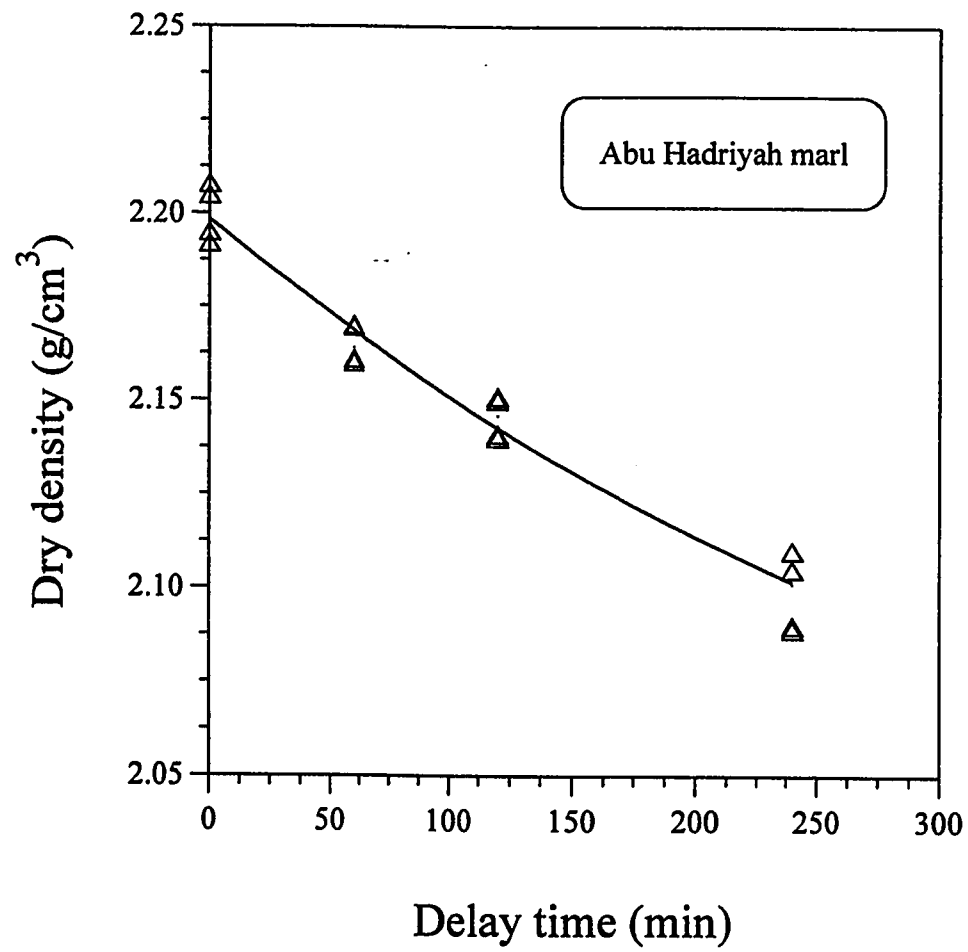


Figure 4.54: Variation of Dry Density with Delay in Compaction for M-ABH9.

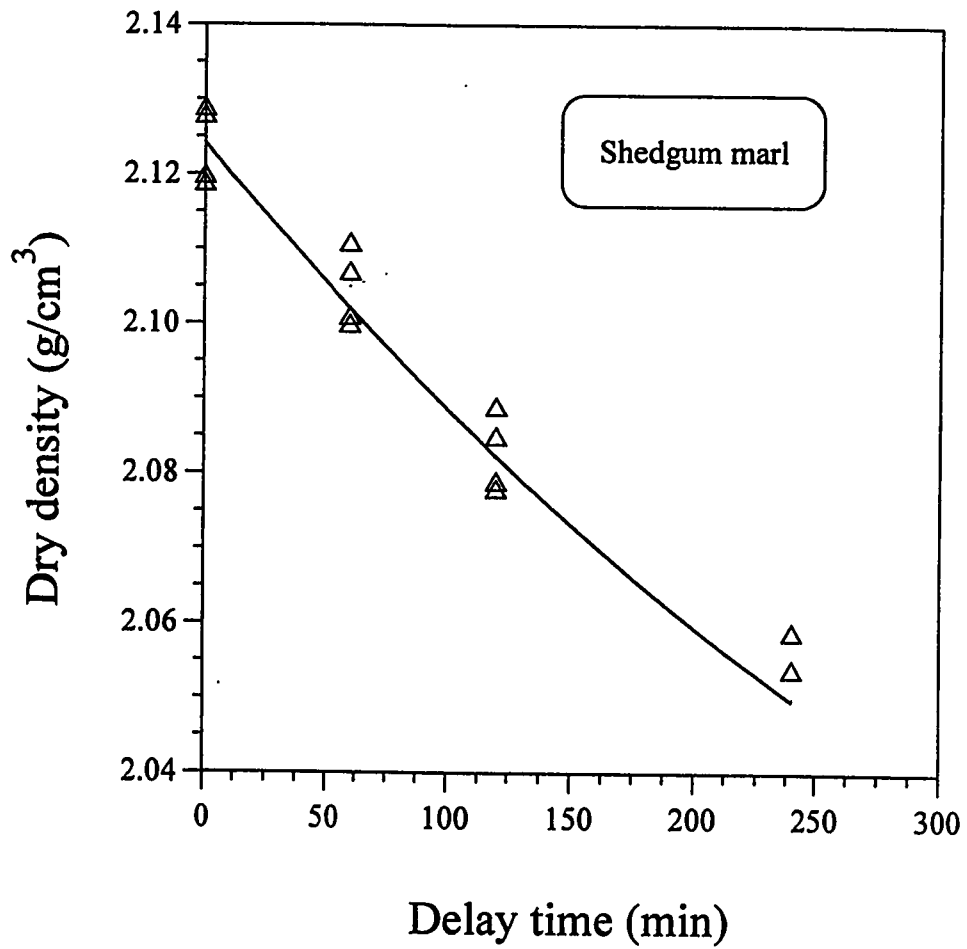


Figure 4.55: Variation of Dry Density with Delay in Compaction for M-SHD1.

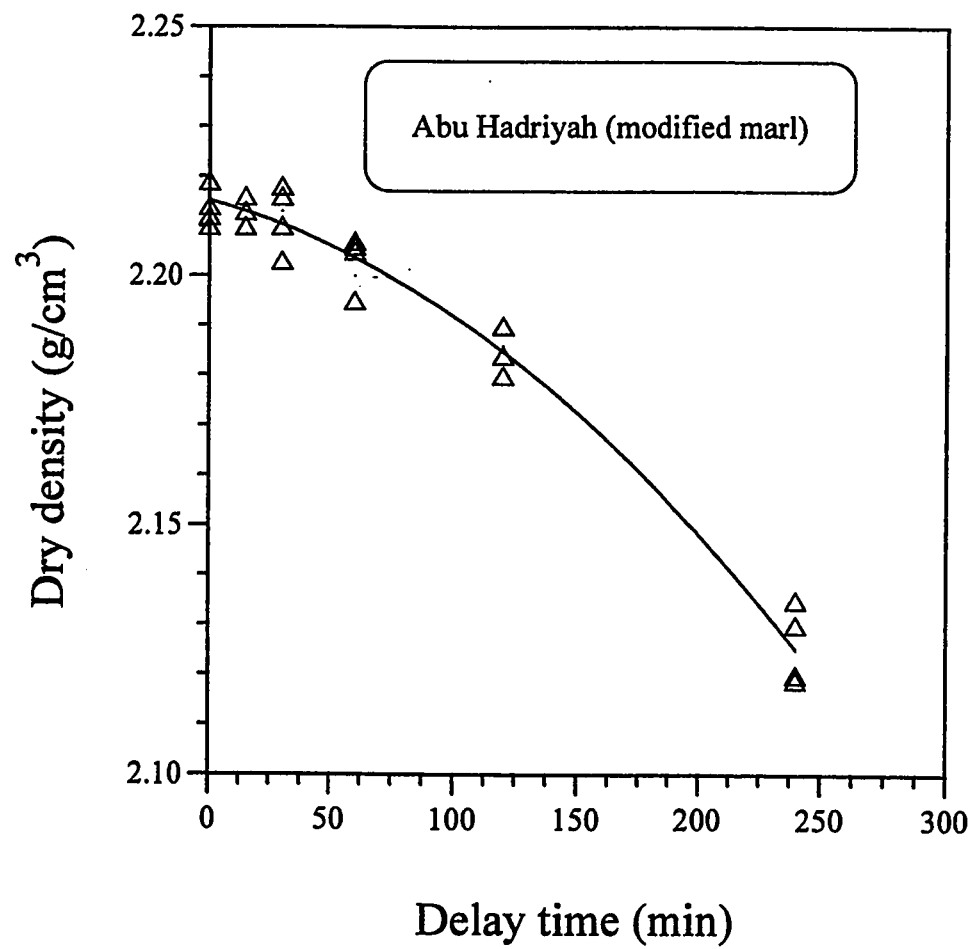


Figure 4.56: Variation of Dry Density with Delay in Compaction for M-ABH9a.

there is a reduction in water content of the soil-cement mixtures that were delayed for certain periods even though evaporation loss was prevented by placing the soil-cement mix in several plastic bags and sealing them completely. Therefore, the reduction in moisture content could be attributed to cement hydration. This reduction affected the compaction process and resulted in some loss of dry density.

In the case of exposed samples, there was a continuous decrease in strength with delay in compaction. The dry density, in addition to other factors, is controlling the moisture evaporation. Due to the decrease in dry density with delay, the reduction in strength of exposed samples is enhanced. It is clearly seen that the positive factors contributing towards the strength in the case of sealed samples, were not as effective as in the case of exposed samples.

To counteract the reduction in strength and density due to delay in compaction, the use of a set-retarder for cement can be an effective solution. For this purpose, 0.5% sugar by weight of cement was used for the one hour delayed samples made with M-ABH9a soil and which were sealed at 21°C for 7 days. The density achieved was even higher than that of the samples compacted with no delay. Similarly, the strength was quite high and even higher than the other samples with one hour delay, as shown in Fig. 4.51. The set retarder delayed the cement hydration and the cement performed like an inert material at the time of compaction. Moreover, the high specific gravity of the cement (3.15) resulted in a higher dry density. The higher strength after 7 days was partly due to the higher dry density and partly due to the cement hydration after compaction thereby resulting in better and effective soil-cement mixture.

Remixing the delayed soil-cement mixtures just before compaction helps improve the strength. This has been observed by remixing some of the mixtures for a period of three minutes and thereafter subjected to one hour delay. Remixing helps in breaking the bonds already formed in the delayed mixtures, thus achieving a high density and consequently a high strength.

Effect of Age

Hydration of cement continues for a long time, however, its rate decreases with time. This process is responsible for the strength gain of cement-stabilized soil with time. Knowledge of the variation in strength gain with time is important in design and construction procedures. It is used in the decision making for the curing methods and periods. In the cases where soil-cement is used in pavements, opening the road for traffic or laying out of the asphalt concrete, is dependent on the strength of the soil-cement layer at the time of opening. Usually 7 day UCS is used for design purposes (Bhatia, 1967), while 3 day strength is used for quality control purposes in field construction. Strength after 28 days can also be used for the capacity evaluation of an existing pavement. However, to speed-up construction, the 24 hour strength is sometimes utilized.

The variation in strength of soil-cement mixtures with time was studied for one of the selected marls (M-ABH9). Sealed as well as exposed samples were prepared and tested after curing periods of 1, 3, 7, 14, and 28 days at room temperature (21°C). The results, shown in Fig. 4.57, clearly indicate the sharp increase in strength of exposed samples; the peak strength was reached in about 14 days.

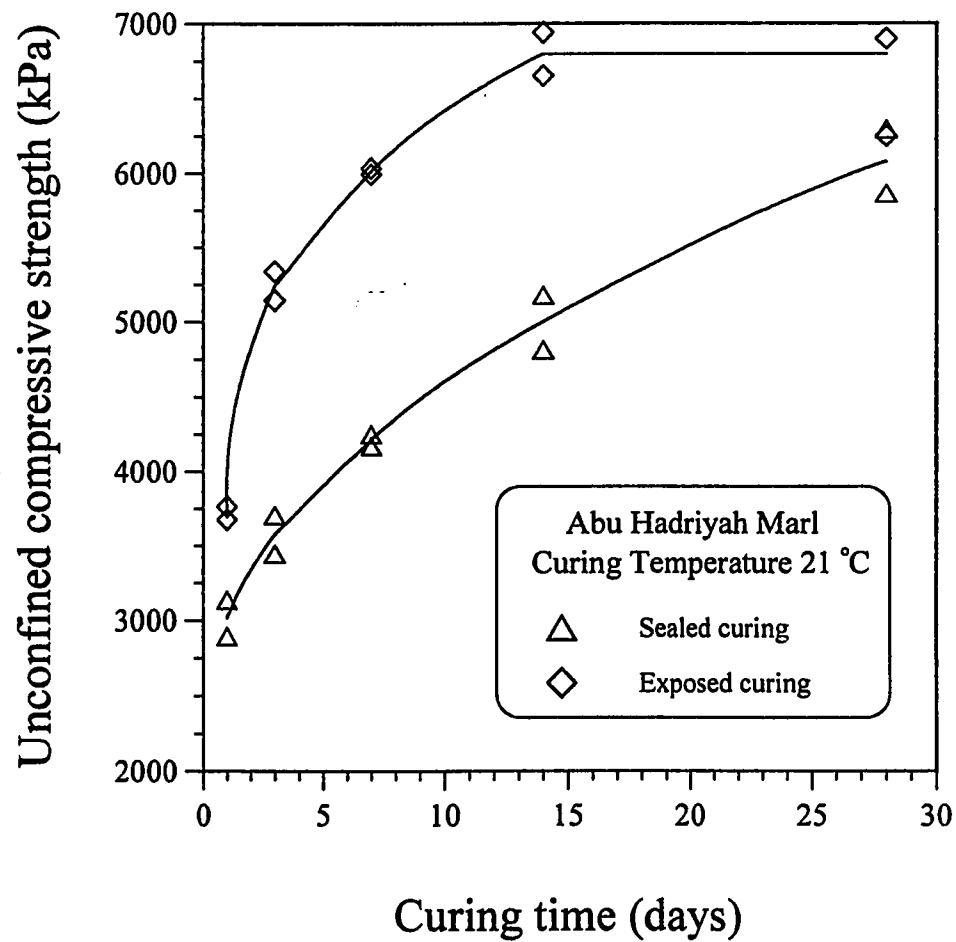


Figure 4.57: Variation of Unconfined Compressive Strength with Curing Period for M-ABH9.

Longer curing period (up to 28 days) did not result in a further increase in the strength. This sharp increase in strength is mainly due to the high evaporation of moisture which resulted in a lower residual moisture content, and consequently carbonate cementation took place. After 14 days, the reduction in moisture content due to evaporation continues and the water needed for cement hydration becomes unavailable. Therefore, the strength showed an almost constant value after 14 days.

On the other hand, the rate of strength gain for the sealed samples was lower in the initial days of curing. However, it reached a slightly lower value compared to that of the exposed samples after a curing of 28 days. Moreover, the strength did not reach a constant value after 28 days, but still show signs of strength increase with time. This was due to the availability of sufficient moisture for the hydration of cement at all stages of curing. Table 4.7 shows that for sealed samples, the residual moisture content was 4.8% for the samples tested after 28 days of curing.

4.6.2 Resilient Modulus

The diametral resilient modulus test was adopted to investigate the behavior of stabilized marls under dynamic loading. Samples were subjected to various curing conditions. Resilient modulus values of sealed versus exposed samples for both marls (M-ABH9 and M-SHD1) at different temperatures are shown in Figs. 4.58 and 4.59. The effect of compaction delay on the resilient modulus values was investigated for two hours delay; Figs. 4.60 and 4.61. Other exposed samples were tested after a curing period of 7, 28 and 56 days; Figs. 4.62 to 4.63.

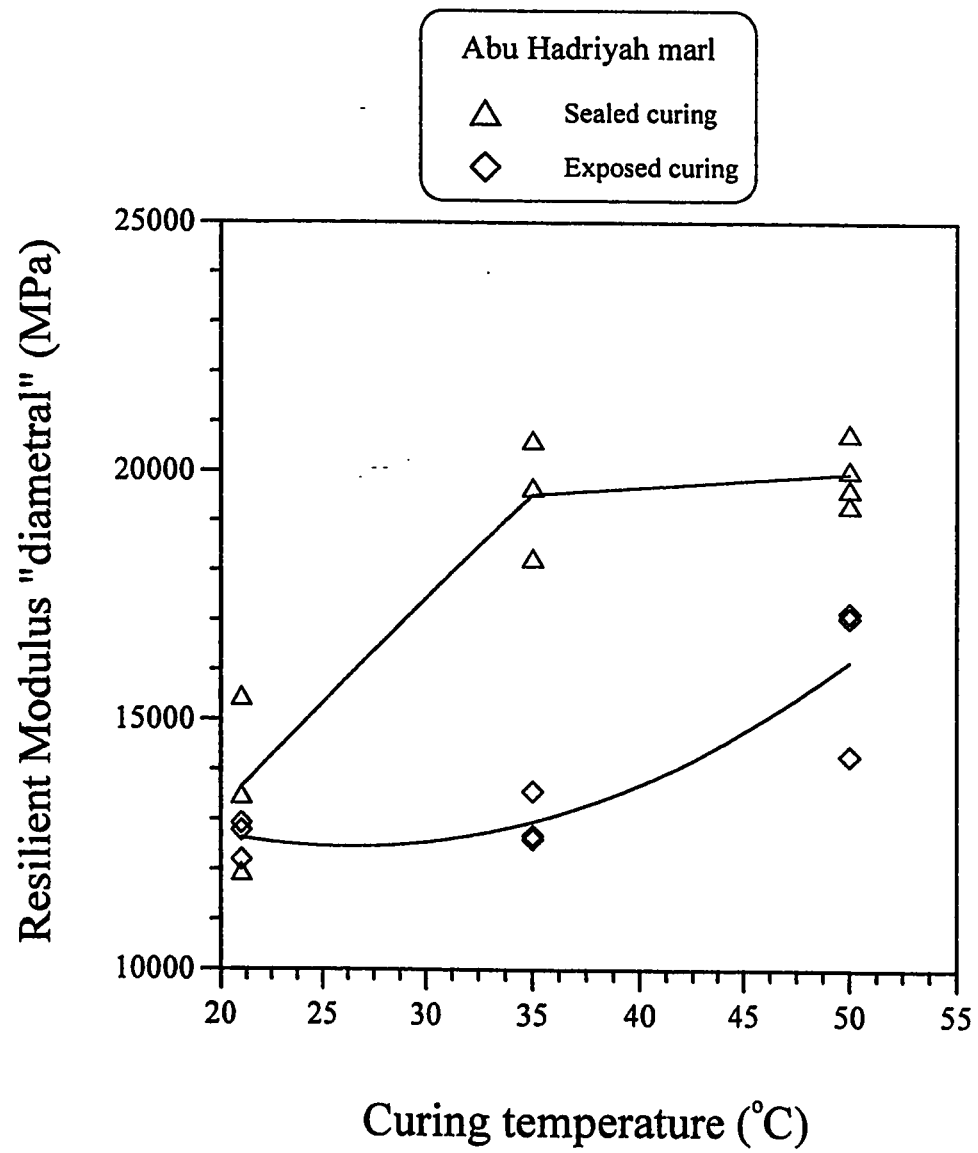


Figure 4.58: Variation in Resilient Modulus with Curing Temperature for M-ABH9 Treated with 5% Cement.

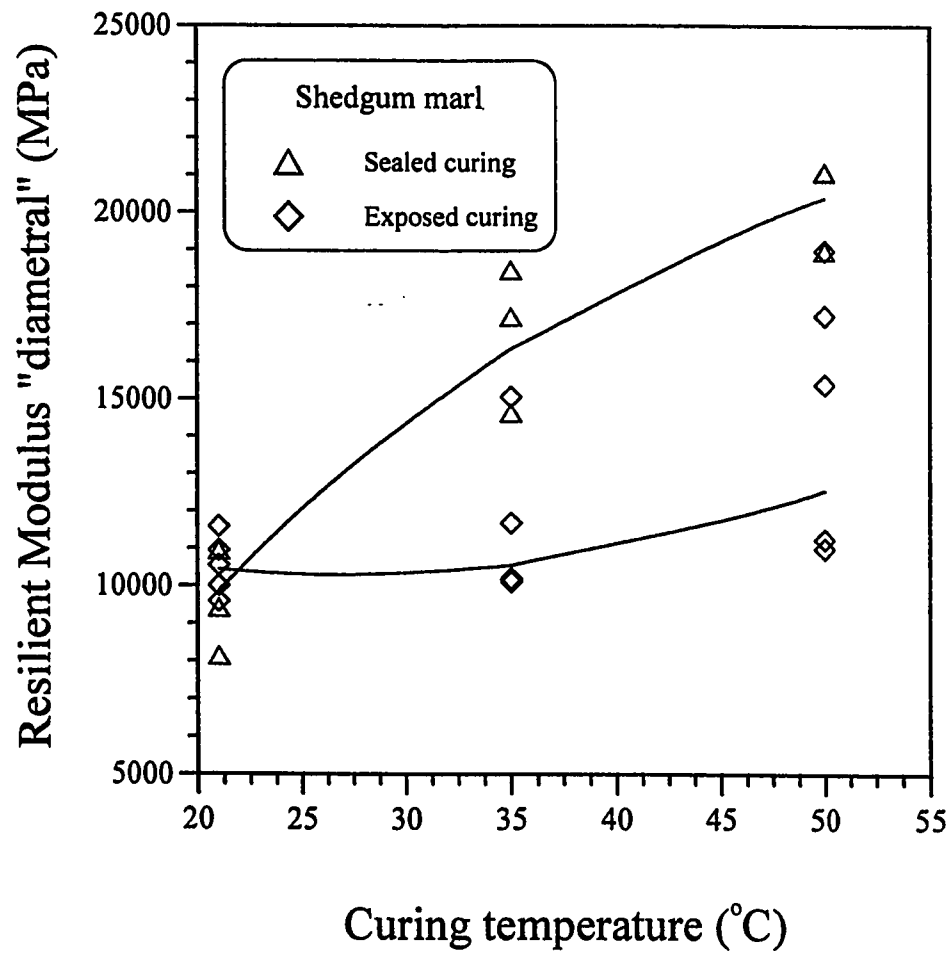


Figure 4.59: Variation in Resilient Modulus with Curing Temperature for M-SHD1 Treated with 5% Cement.

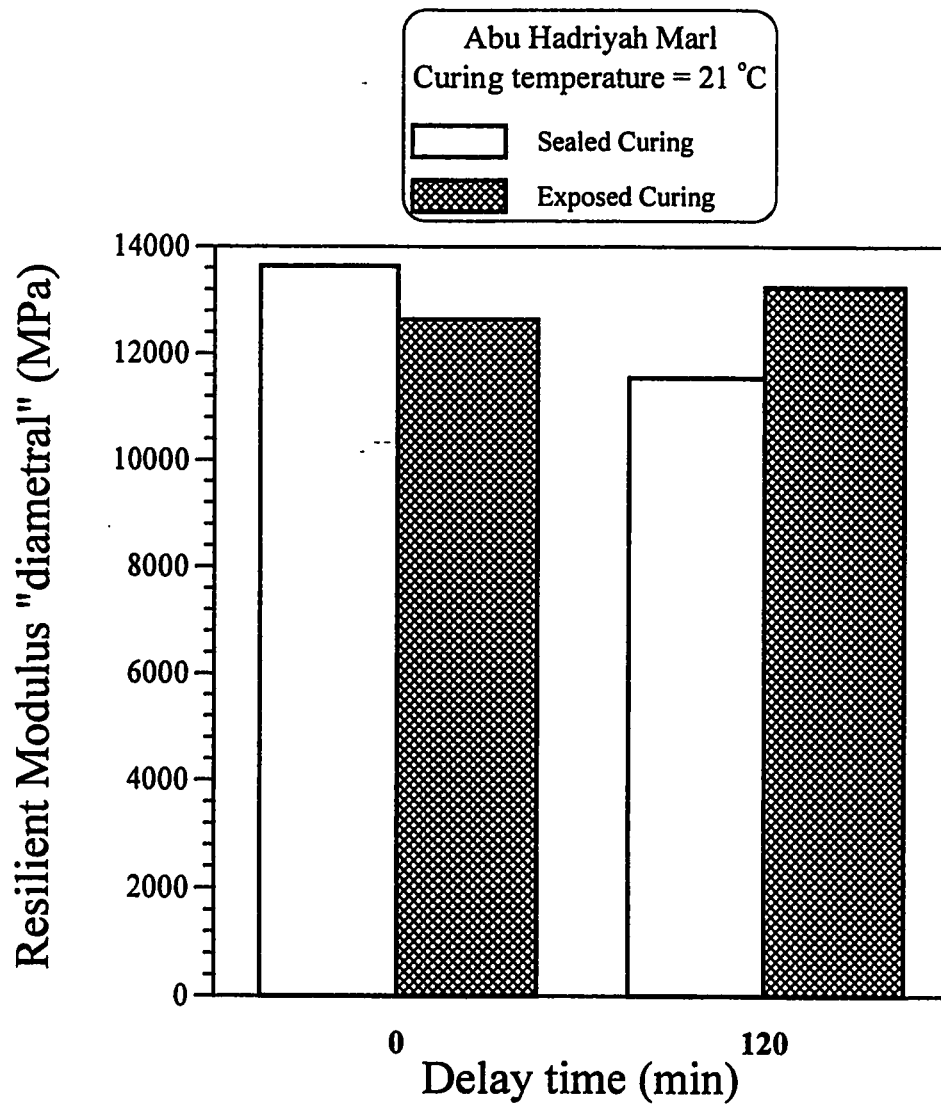


Figure 4.60: Effect of 2 Hours Delay in Compaction on Resilient Modulus for M-ABH9 Treated with 5% Cement.

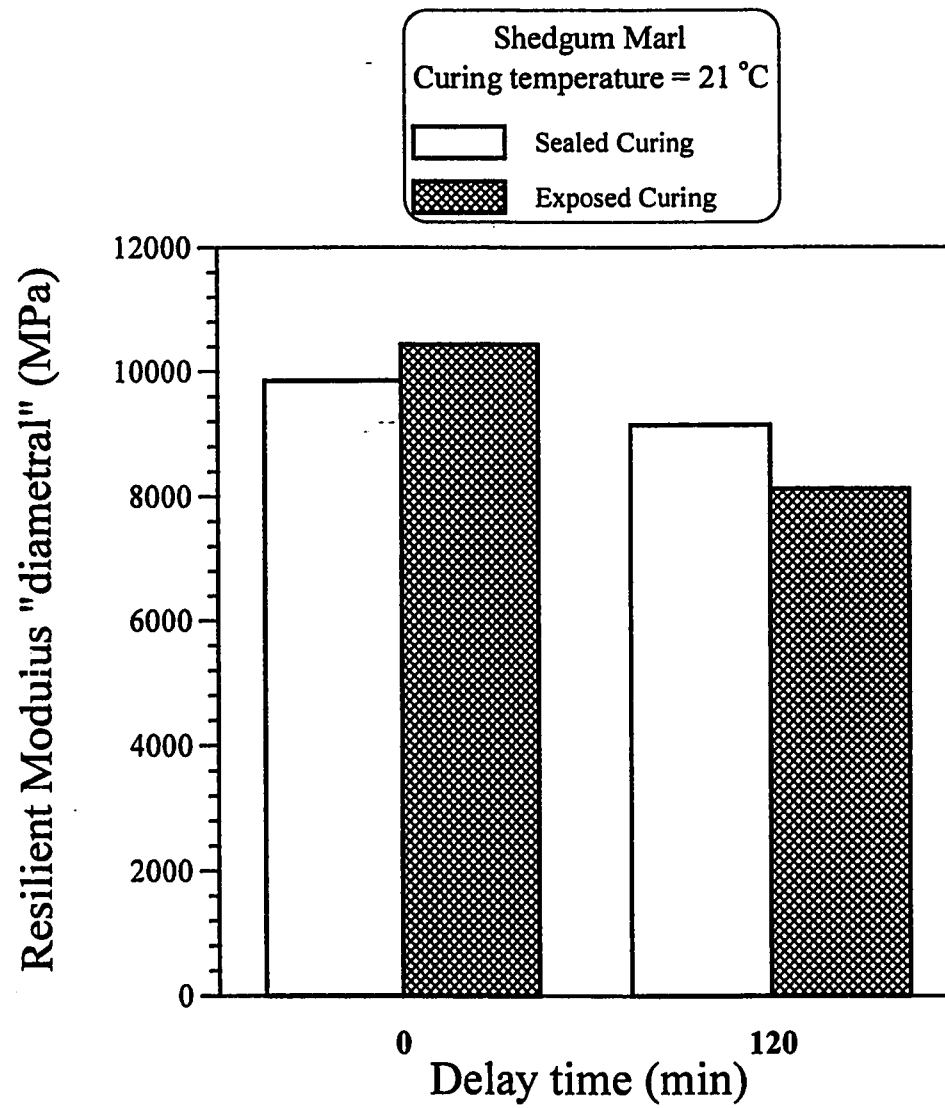


Figure 4.61: Effect of 2 Hours Delay in Compaction on Resilient Modulus for M-SHD1 Treated with 5% Cement.

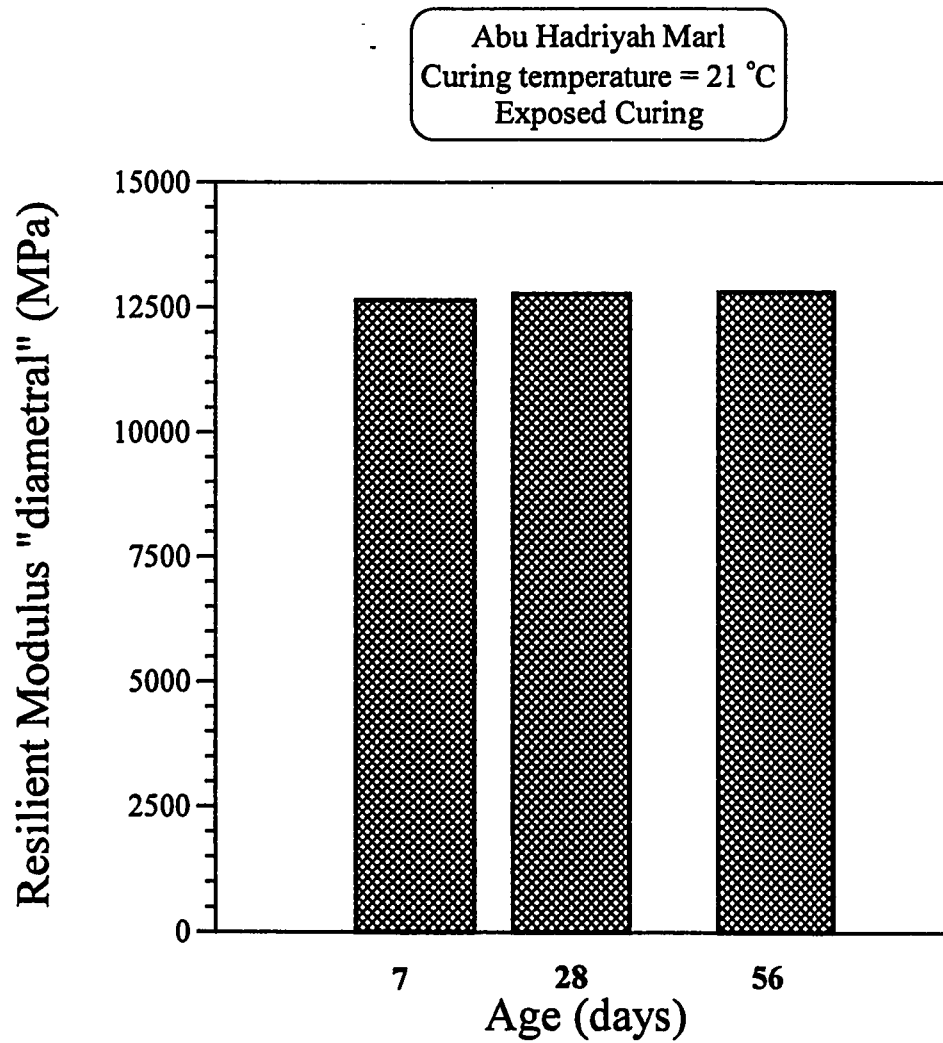


Figure 4.62: Effect of Age on Resilient Modulus for M-ABH9 Treated with 5% Cement.

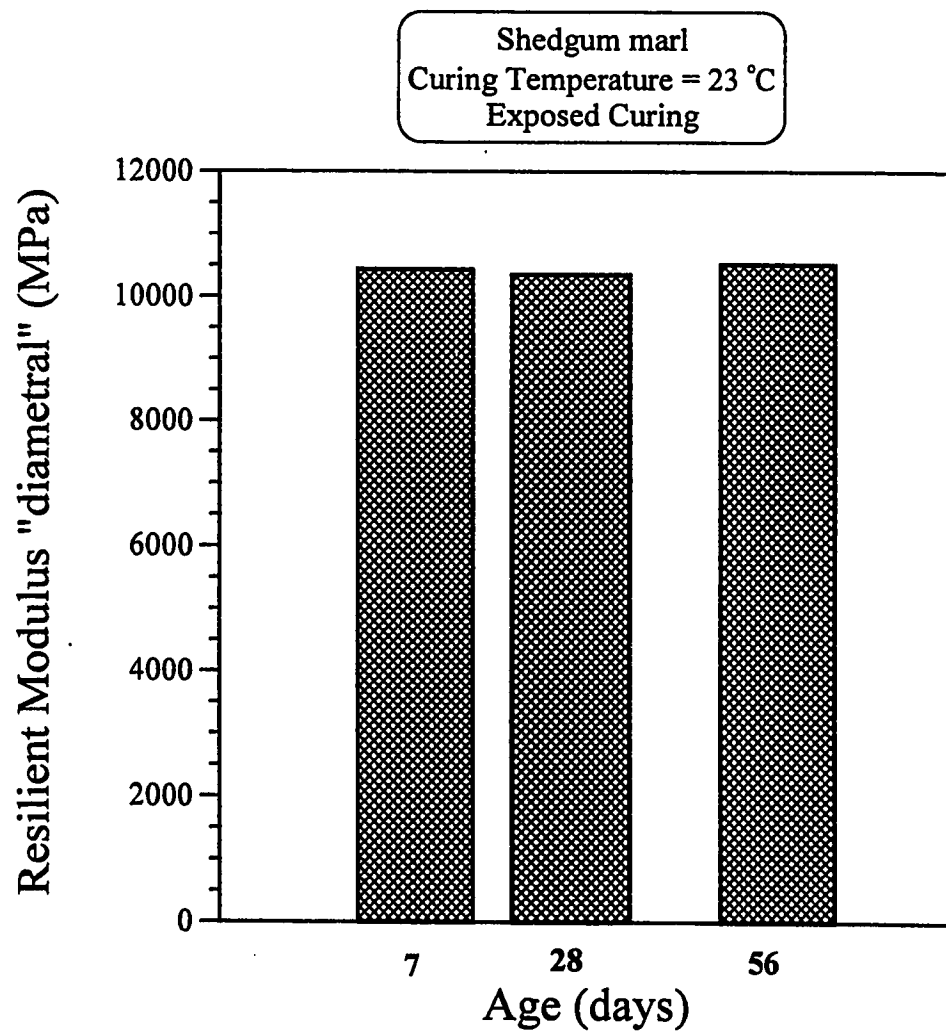


Figure 4.63: Effect of Age on Resilient Modulus for M-SHD1 Treated with 5% Cement.

The variation in resilient modulus (M_R) with the curing temperature, and compaction delay for sealed and exposed samples is almost similar for both marls. The trend is not the same for the delayed samples; the sealed samples exhibited higher modulus values for one of these marl, while the exposed ones showed higher values for the other marl. This behavior may be soil dependent; carbonate cementation is helpful in one case while it is not in other case. Moreover, the four data points for each curing condition are overlapping with each other. So, these trends cannot be generalized in the absence of sufficient data.

Although the M_R for M-ABH9 is a little high, the resilient modulus for the samples cured at room temperature (21°C) is almost independent of the curing regime. Increase in the curing temperature caused an increase in M_R for the sealed samples, while no significant increase in M_R was observed in the case of exposed ones. This behavior is not consistent with that of the unconfined compressive strength results. Further, it is clear from these figures that the exposed samples are slightly affected by the curing temperature, and the resilient modulus values for both marls are constant up to 35°C , and increased slightly when the curing temperature was raised to 50°C . A slow-forming strong cement bond in the case of sealed samples resulted in lower resilient or recoverable strains during testing. However, the exposed samples lacks this strong bonding and the samples become even weaker at higher curing temperatures. Higher curing temperatures do ensure higher rates of setting and hardening of cement, but the evaporation of moisture occurs simultaneously in the case of exposed samples. Almost equal contribution of the positive and the negative factors is the probable reason for getting nearly a

constant resilient modulus at different curing temperatures in the case of exposed samples.

Resilient modulus of the exposed samples at room temperatures was found to be almost constant after 7 and up to 56 days of curing. Therefore, the 7 days M_R value can be considered as an ultimate value for these curing conditions. This behavior can be explained in relation to the stress level, the samples are subjected to, during resilient modulus testing. The portion of the stress-strain curve used in resilient modulus testing develops after a few days of curing, further increase in curing time does not affect this portion.

The above discussion reveals an important fact that should be considered in the design and construction of the pavement systems. Sealed curing must be adopted to have a material with a better resilient response. Sealing the cement-stabilized layer should be given further attention when the field temperature is high, as in the summer season. Further, the curing conditions which are responsible for the high UCS in the laboratory result in lower resilient modulus values. Since the resilient modulus reflects the behavior of pavement layers more closely than the unconfined compressive strength, the conditions and factors favorable for resilient modulus should be satisfied in the field.

4.6.3 Grain Crushing

In this investigation, a comparison was made between the effects of standard Proctor, the modified Proctor, and the static compaction. The static compaction was performed at moisture content and dry density corresponding to the modified

Proctor test results. For standard Proctor, the samples were compacted at moisture and dry density corresponding to the results obtained from the same test. However, the grain-size distribution after the tests was compared with the original gradation using wet sieve analysis.

For the different compaction methods and molding moisture contents, various grain-size distributions (GSD) for M-ABH9 were obtained on the same graph. To study the effect of method of compaction on grain crushing, the GSD for the samples compacted at optimum, dry of optimum and wet of optimum moisture content were plotted in Figs. 4.64 to 4.66, respectively. Each plot depicts GSD curves for the original soil, soil after compaction, and a specified limit. GSD for a soil after compaction was obtained for three types of compaction; standard Proctor, modified Proctor, and static compaction. The dashed lines represent the gradation limits specified by the Dammam municipality. The wet and dry side moisture contents correspond to 95% of the maximum dry density. Quantitative analysis of crushing due to compaction is shown in Table 4.8. Percent passing certain sieves after compaction is compared with those of the soil without compaction so as to be used in the AASHTO and USCS soil classification methods. In Table 4.8, a comparison is also made of the median size (D_{50}) of the soil, before and after compaction.

The effect of molding moisture content on the grain crushing for each compaction method is given in Figs. 4.67 to 4.69. The results indicate clearly that grain crushing occurred during compaction and the extent of crushing depends on the compaction method and the molding moisture content. The energy applied

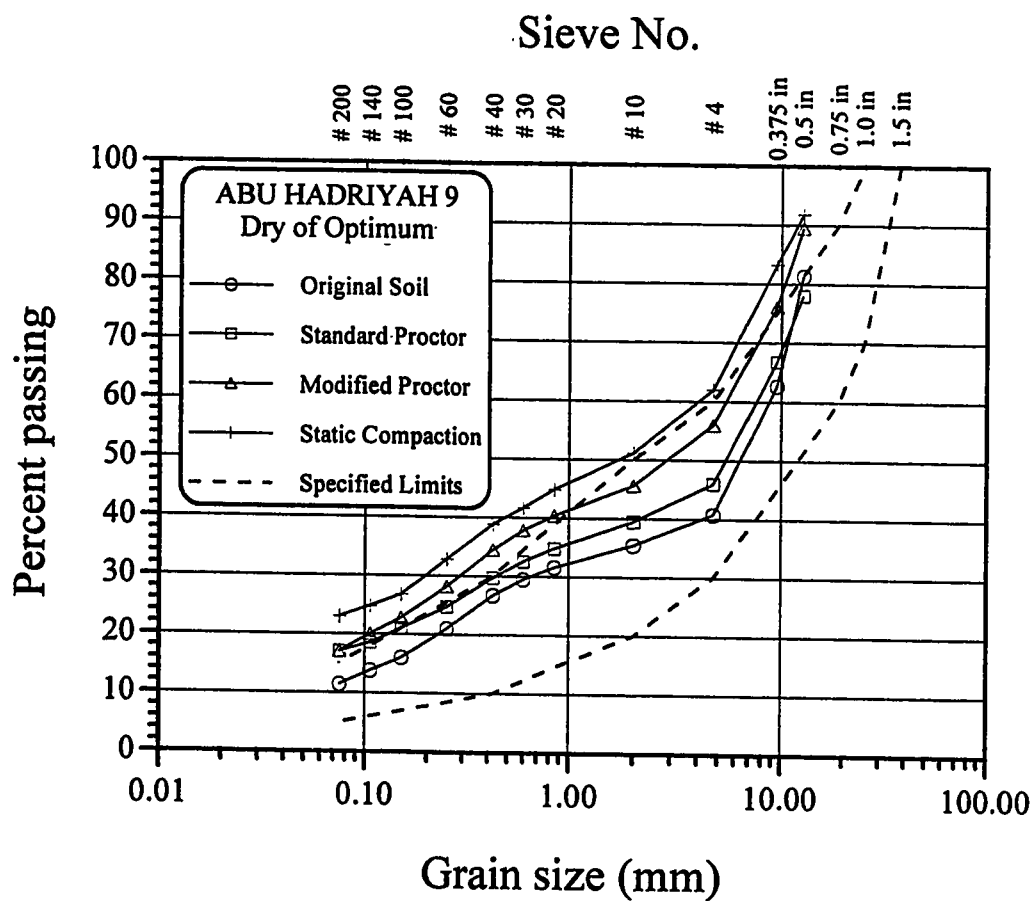


Figure 4.64: The Effect of Compaction Method on the Grain-Size Distribution for M-ABH9 Compacted on the Dry Side of Optimum Moisture Content.

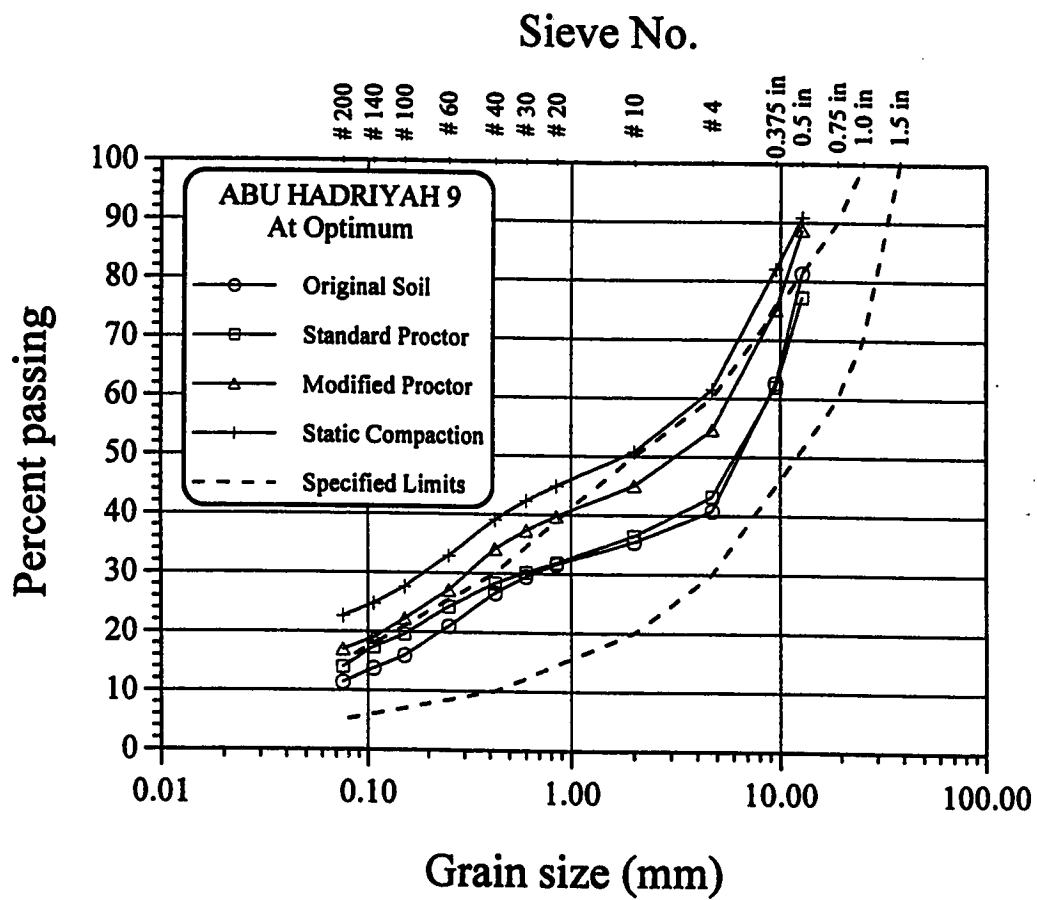


Figure 4.65: The Effect of Compaction Method on the Grain-Size Distribution for M-ABH9 Compacted at Optimum Moisture Content.

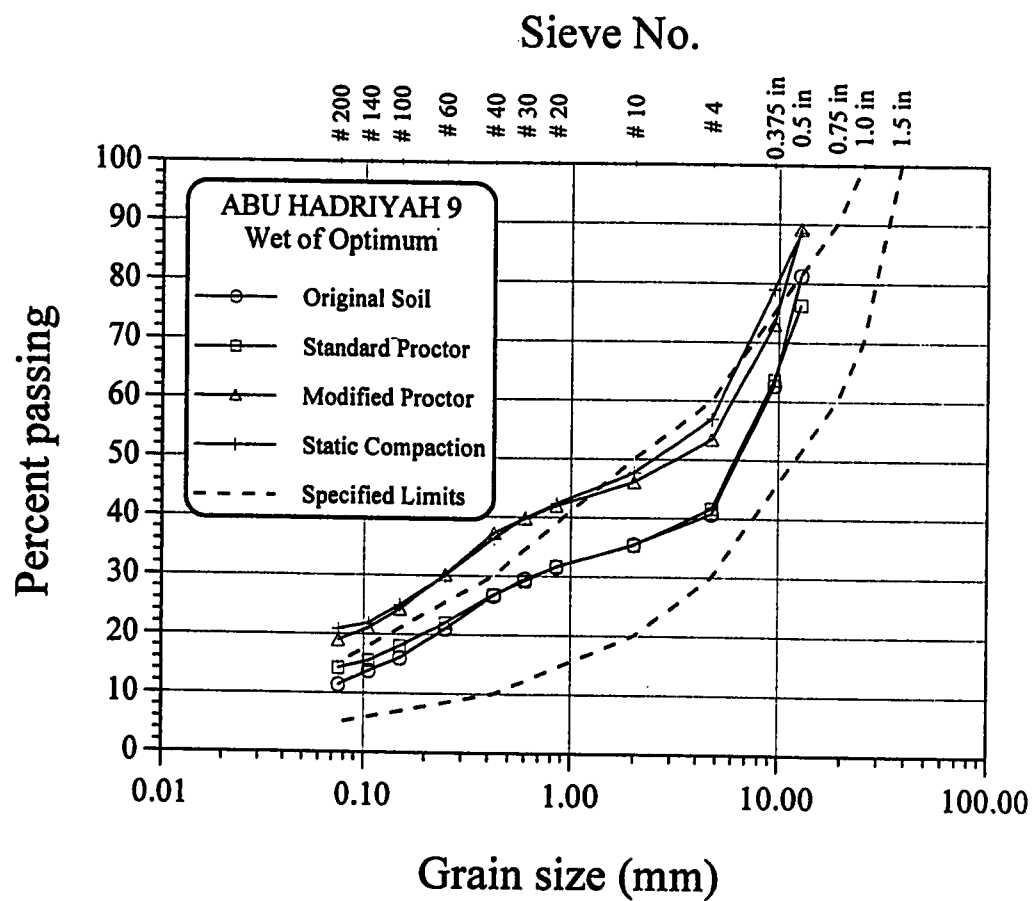


Figure 4.66: The Effect of Compaction Method on the Grain-Size Distribution for M-ABH9 Compacted on the Wet Side of Optimum Moisture Content.

Table 4.8: Effect of Grain Crushing Due to Compaction

Compaction Method	Compacted at	Percent Passing ASTM Sieve				Median Size (D_{50}^{**})	Ratio ^s
		#4	#10	#40	#200		
Before Compaction		40.7	35.4	26.7	11.4	6.0	1.00
Standard Proctor	Dry of Optimum*	46.0	39.5	29.7	17.1	5.1	0.85
	Optimum	43.2	36.4	28.0	14.0	6.0	1.00
	Wet of Optimum*	41.7	35.3	26.8	14.2	6.0	1.00
Modified Proctor	Dry of Optimum*	56.0	45.6	34.5	17.0	2.6	0.43
	Optimum	54.6	45.0	34.1	17.0	3.0	0.50
	Wet of Optimum*	53.5	46.1	37.2	18.9	3.0	0.50
Static Compaction	Dry of Optimum*	62.0	51.2	38.8	22.9	1.5	0.25
	Optimum	61.5	50.7	39.1	22.7	1.6	0.27
	Wet of Optimum*	56.9	47.7	36.4	20.8	2.4	0.40

* moisture content corresponding to 95% of the maximum dry density

** grain size coarser than 50% of the material

s ratio of D_{50} after compaction to that before compaction

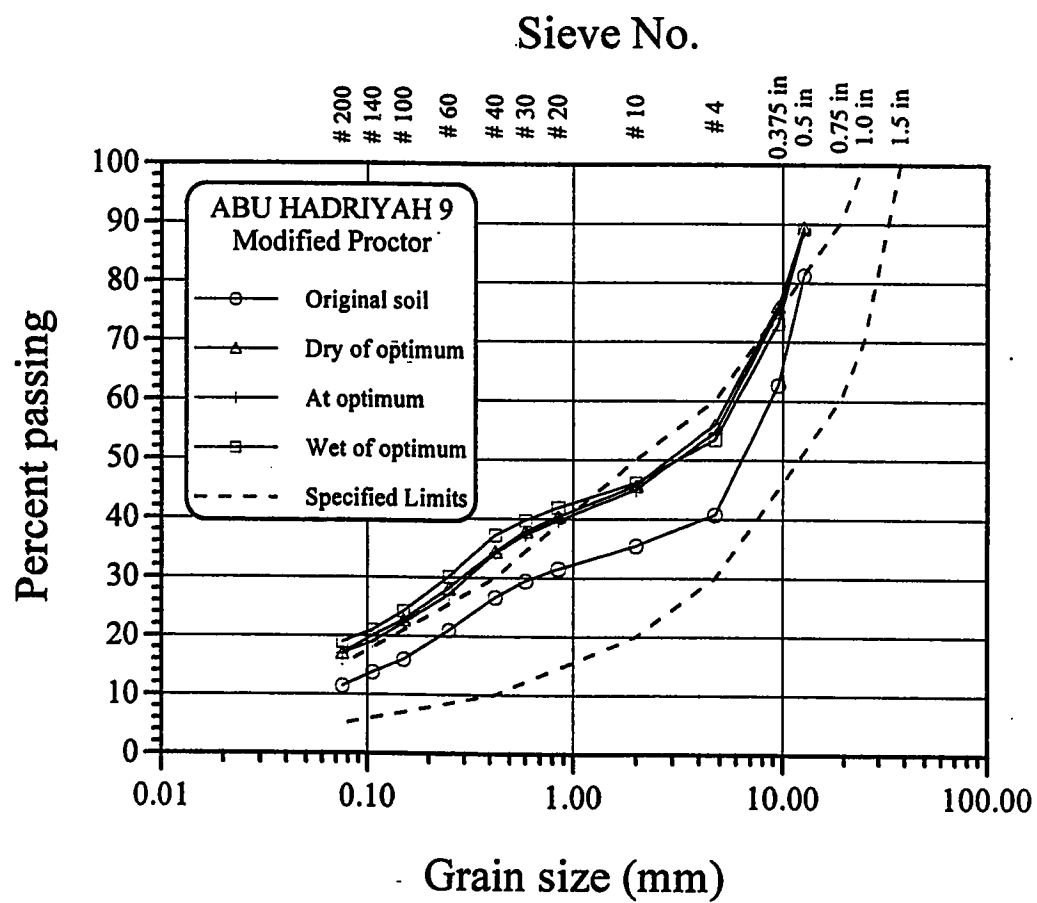


Figure 4.67: The Effect of Molding Moisture Contents on the Grain-Size Distribution for M-ABH9 Compacted Using Modified Proctor.

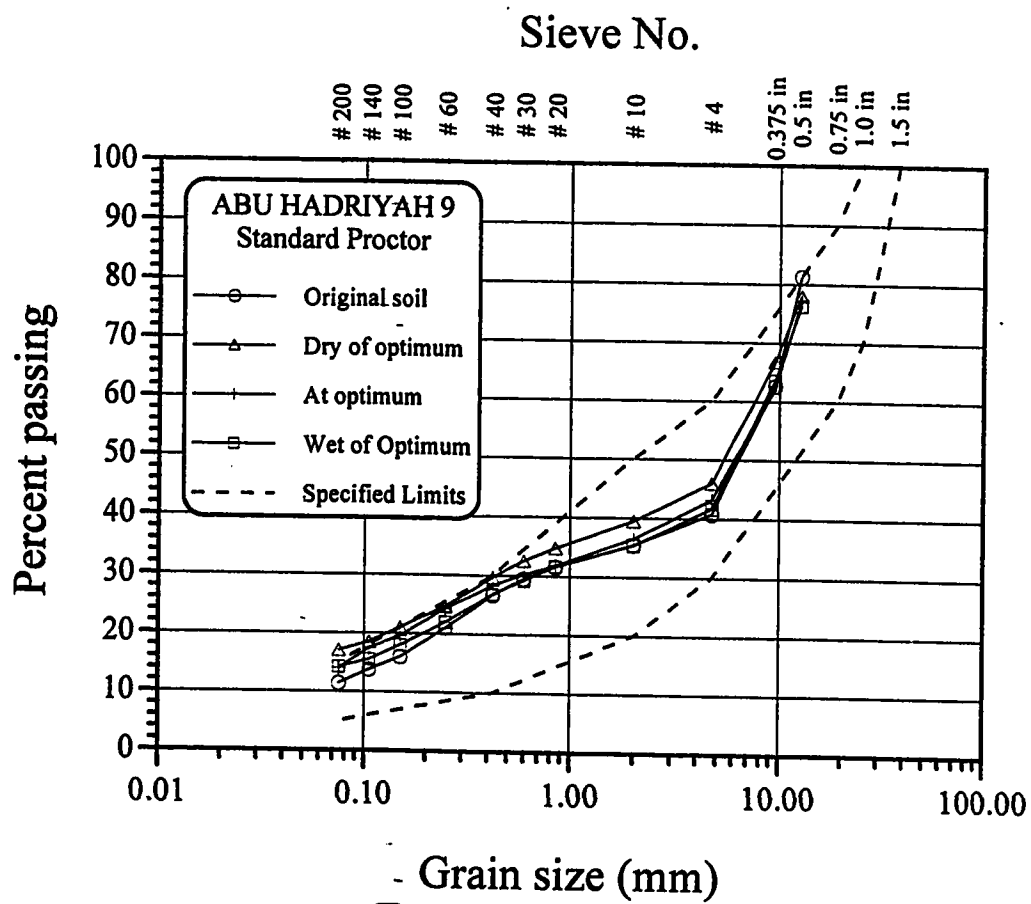


Figure 4.68: The Effect of Molding Moisture Contents on the Grain-Size Distribution for M-ABH9 Compacted Using Standard Proctor.

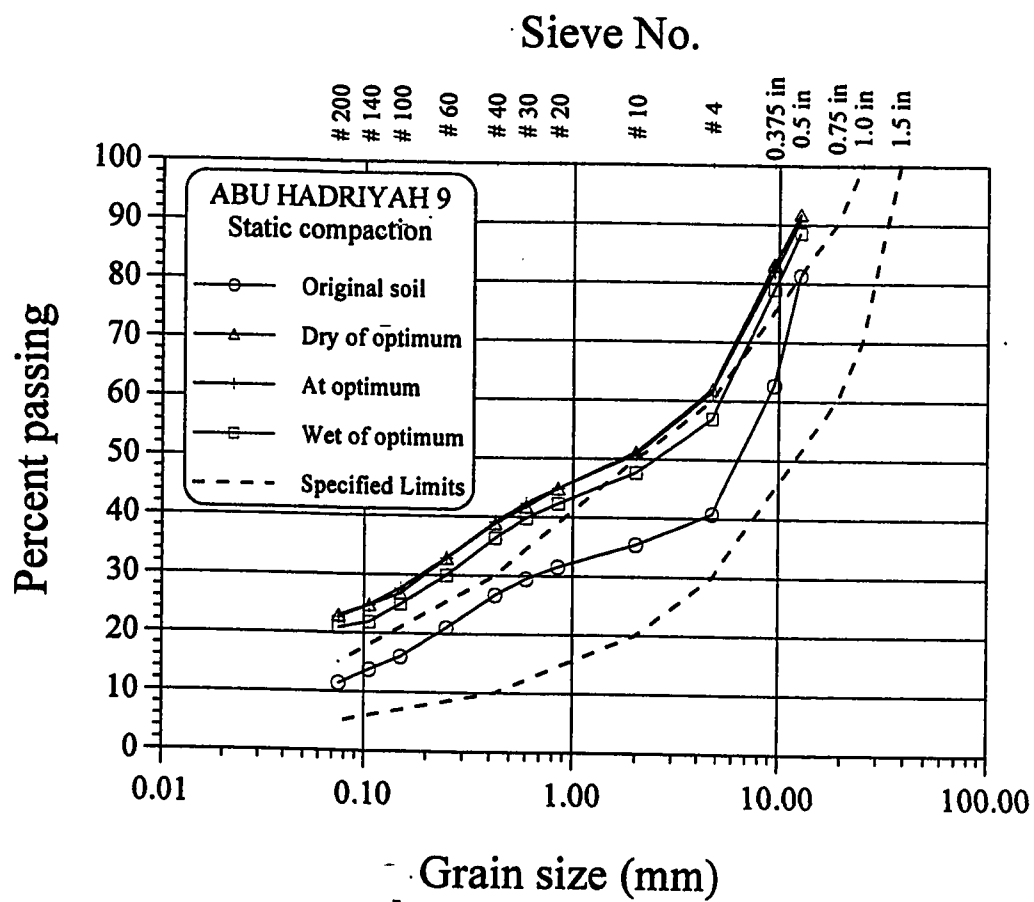


Figure 4.69: The Effect of Molding Moisture Contents on the Grain-Size Distribution for M-ABH9 Compacted Using Static Compaction Method.

in each test causes two actions; compression of soil mass and rearrangement of soil particles. During compression, high contact stresses are generated and considered responsible for the crushing and breakage of soft aggregates. The extent of breakage depends on the magnitude of contact stresses which in turn depends on the dynamic energy. Since the energy input transferred to the soil mass is more in the case of the modified Proctor compared to the standard one, the grain crushing in the case of the modified Proctor is more than that of the standard one.

The crushing of soil particles was maximum in the case of static compaction. This was mainly due to the difficulty of the particles' rearrangement during compaction. Since the diameter of the compression piston is almost the same as that of the mold, the particles could not easily rearrange themselves to a closer packing, rather the whole soil mass was compressed to a higher density. Also, lack of vibrations contribute to the inability of soil particles to get rearranged. Hence, very high contact stresses were generated and responsible for the relatively greater crushing when compared to the other "dynamic" methods. In other words, the required density was mostly achieved at the cost of particle crushing.

The molding moisture content contributes to the grain crushing through lubrication. It is clear from the plots that minimum crushing took place for samples compacted on the wet side of optimum for all compaction methods, except for modified Proctor test. This may be due to the variability in the gradation of the original sample, each curve being the average of the gradation of replicate samples. However, at the optimum and dry of optimum moisture content, the extent of crushing was almost the same. The reason for the lesser crushing in the case

of samples compacted on the wet side is the enhanced lubrication effects, ease of rearrangement and lower shear strength. Lower shear strength is due to higher pore pressures, resulting in decrease in contact stresses, and consequently crushing. Therefore, the energy transferred to the soil mass was not resisted much, and the particle arrangement became easy. The pore water pressure is much less in the case of samples compacted at optimum and on the dry side of optimum. These reasons were responsible for the mitigated crushing of particles on the wet side of optimum.

On the other hand, greater crushing took place in the case of samples compacted at the optimum and dry of optimum due to the lack of lubrication. When the compaction energy is applied, the particles will try to rearrange themselves but a higher resistance is offered at the contacts thus causing the grain to crush. In addition, the macropeds are larger at lower moisture contents (at both the optimum and dry side of optimum), thus the number of contacts will be less than those in the case of samples compacted on the wet side of optimum and that intensifies the contact stresses, and enhances grain crushing. Another probable reason might be the crushing during the initial stages of compaction, which was almost the same for the samples compacted on the dry side of the optimum and at the optimum. According to Sandford et al. (1980), the maximum crushing of particles takes place in the initial stages of compaction. As the density increases to a certain level, the crushing is reduced. Later on, rearrangement of particles to a denser packing would result in lesser crushing. The increase in density usually results in an increase in the number of contacts thereby ensuring lower contact stresses.

Comparing the grain crushing results with the specifications, it is noted that the gradation after compaction may not meet the specified gradation. The gradation limits required by the Dammam municipality is only met by the gradation before compaction, Figs. 4.64 to 4.69. After compaction, the gradation obtained even from the standard Proctor test is on the margin of fine limit gradation. The gradations resulted from the modified Proctor and static compaction tests are also crossing the ASTM gradation bands A, B, and C, Fig. 3.6, especially with regards to the fines passing ASTM No. 200 sieve. Moreover, the fine gradation resulting from the compaction changes the AASHTO classification of the soil from A-1-a to A-1-b.

Chapter 5

Summary, Conclusions and Recommendations

5.1 Summary

Calcareous soils are considered abnormal in their behavior. Marl, a type of calcareous soils, is extensively found in the Eastern Province of Saudi Arabia. The lack of other good quality materials enhanced the extensive exploitation of calcareous sediments in base construction of roads and foundations. The heterogeneous nature of calcareous soils and the lack of published information on the behavior of these soils under the harsh environmental and loading conditions have led to speculation and discrepancies in the qualification of them as a construction material. Many post construction problems have been reported, such as the complete failure of roads, few month after construction, when marl soil was used as a base material. In addition, the regular classification systems and specifications do not hold true for these soils.

Determination of the properties and the feasible solutions were the primary objectives of this study. Marl samples were collected from over 20 different sites in

the Eastern Province of Saudi Arabia. The samples were characterized according to AASHTO and USCS classification systems as well as the moisture density-CBR relationships. Based on the results of these tests, it was found that marl soils are extremely sensitive to the molding and testing (i.e., final) moisture contents. Owing to their inferior performance as construction materials, chemical stabilization of selected marl soils was undertaken. Cement stabilization was found to be the most suitable and economical way of utilizing these 'low quality' soils.

5.2 Conclusions

In light of the comprehensive experimental study conducted in this thesis on calcareous "marl" soils, the following conclusions can be drawn:

- Most calcareous soils do not conform to the definition of 'marl' due to the absence of clay. These soils can be termed "carbonate soils", "calcitic soils", or "dolomitic soils" depending on their dominant mineral(s).
- Calcareous soils in eastern Saudi Arabia exhibit significant variations in their constituents. The variability exists not only from place to place but also in the lateral direction and in the different layers at the same place.
- Although the calcareous soils were classified as "excellent" according to the classification systems, they exhibited unsatisfactory behavior when used as construction materials. Therefore, there is a need for a modified system which may include chemical and mineralogical tests to judge the type and nature of materials. CBR (soaked and unsoaked) should be included in the characterization tests to quantify the effect of soaking.

- Most of the calcareous soils are extremely sensitive to molding as well as testing moisture contents. There is complete loss of bearing strength upon the increase in the molding moisture content beyond the optimum moisture content.
- Portland cement was proven to be a suitable chemical additive to stabilize eastern Saudi calcareous soils.
- A cement content of 5% was found to be adequate for the effective stabilization of local calcareous soils. It meets the the strength and the durability requirements.
- Resilient modulus (M_R) values increased with the increase in curing temperatures for sealed curing. For the exposed curing conditions, however, the M_R values were not affected much by the increase in temperature. M_R at 7 days, when cured under exposed conditions at 23°C, can be considered an ultimate value for these curing conditions.
- The unconfined compressive strength of cement-marl mixtures increases with an increase in the curing temperature. At higher curing temperatures, however, control of moisture loss becomes more important, especially if there is a delay in compaction.
- Delay in compaction reduces both the dry density and strength. However, if the moisture loss is prevented, properly cured soil-cement mixtures exhibited an increase in strength within one hour of delay. To counteract the effects of delay, a set-retarder in an appropriate dosage can be used.

- When marl soils are used without improvement, crushing of particles will take place during compaction and loading conditions. The gradation thereafter may not meet the required gradation limits.
- Slake durability is proven to be a good alternative to the existing ASTM D 559 durability standard. Although the results of both tests were in close agreement, the consistency and accuracy of performing the slake durability tests are of great advantages over the ASTM D 559 test.

5.3 Recommendations for Further Study

- There is a need to develop a soil classification system for calcareous sediments. The system should be based on simple and easy tests, and should be able to predict the field performance.
- Self-stabilization of calcareous soils are ascribable to the deposition and recrystallization of the carbonates on the soil grains' contacts. The factors affecting the self-stabilization can be studied in detail thereby utilizing this mechanism positively.
- Mechanical stabilization using different binders can be performed, so as to provide stability to the soil-binder mixtures.
- Gradation might be another parameter affecting the strength and stability of these soils. This can be studied using different gradations for the same soil and their effects on the strength, stability and grain crushing should be investigated.

- The effect of salinity of the molding water can be studied using waters with different salinities in the mixing process.
- Micro-level studies can be performed to investigate the nature and type of bonds formed due to various effects, such as cement content, moisture, temperature, delay in compaction, etc.
- Field trials should be executed to confirm the laboratory findings. With the feed back data of these trials, specific recommendations and modifications can be positively stated.

Nomenclature

AASHTO	American Association for State Highway and Transportation Officials
ACI	American Concrete Institute
A.F.V	Aggregate Finger Value (%)
A.P.V	Aggregate Pliers Value (%)
ASTM	American Society for Testing and Materials
cmc	Compaction Moisture Content (%)
C_c	Compression Index
C_r	Rebound Index
C_s	Swelling Index
CBR	California Bearing Ratio (%)
CIU	Isotropically Consolidated Undrained Triaxial Shear Strength Test
CIV	Clegg Impact Value
CSS-1h	Cationic Slow-Setting Emulsified Asphalt
D_{10}	Effective Grain Size, Grain Size Corresponds to 10% of the Soil Sample Passing by Weight
D_{50}	Median Grain Size, Grain Size Corresponds to 50% of the Soil Sample Passing by Weight
E	Modulus of Elasticity (MPa)
e	Void Ratio
EDTA	Ethylenediaminetetracetic Acid
GE	Calcrete Gravels for Which More Than 40% Passing 0.4 mm Sieve
GSD	Grain Size Distribution
S_c	Susceptibility of Crushing
LL	Liquid Limit (%)
LVDT	Linear Variable Differential Transducer
M	Molar
mM	Milli Molar
MC-70	Medium Curing Cutback Asphalt
MOC	Ministry of Communication
M_R	Resilient Modulus (MPa)
OMC	Optimum Moisture Content (%)
PI	Plasticity Index (%)
PL	Plastic Limit (%)

rmc	Residual Moisture Content (%)
SE	Calcrete Sands for Which More Than 40% Passing 0.4 mm Sieve
SEM	Scanning Electron Micrograph
SL	Shrinkage Limit (%)
UCS	Unconfined Compressive Strength (kPa)
UCT	Unconfined Compression Test
USACE	United States Army Corps of Engineers
USCS	Unified Soil Classification System
XRD	X-ray Diffraction
ϕ	Angle of Internal Friction (degrees)
σ_d	Repeated Axial Deviator Stress (MPa)
γ_{dmax}	Maximum Dry Density (g/cm ³)
ϵ_a	Recoverable Axial Strain (%)

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Vita

- Habib-ur-Rehman Ahmed
- Born in Sargodha, Pakistan
- Received Bachelor of Science Degree in Civil Engineering from the University of Engineering and Technology, Lahore, Pakistan in September, 1992.
- Worked as Design Engineer with Pak Consolidated Consultants (PCC), Lahore, Pakistan from February, 1993 to May, 1993.
- Worked as an Engineer with Water And Power Development Authority (WAPDA), Pakistan from May, 1993 to July, 1993.
- Completed Master of Science Degree Requirements at King Fahd University of Petroleum and Minerals, Dhahran, Saudi Arabia in December, 1995.